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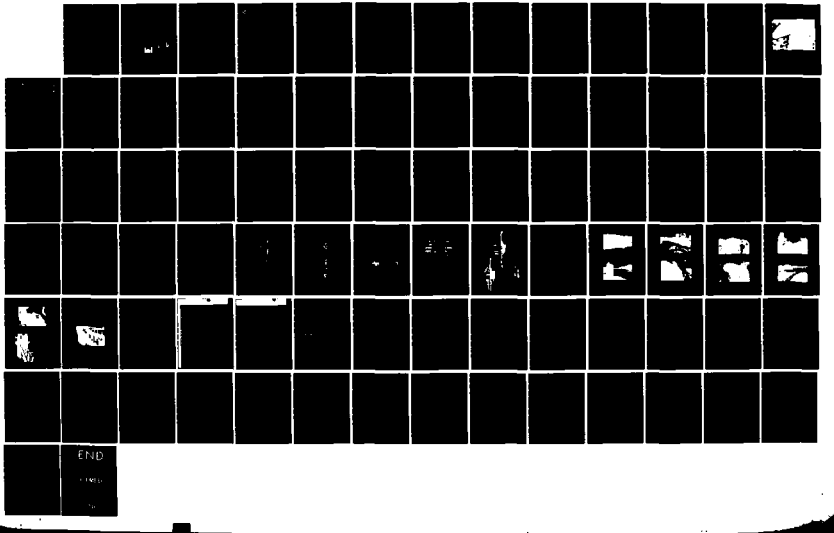

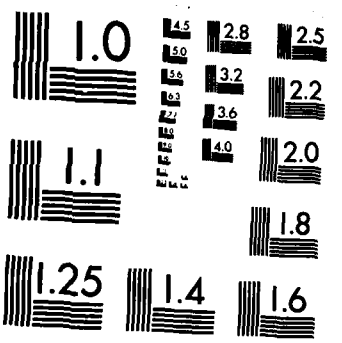

MERRIMACK RIVER BASIN
FRAMINGHAM, MASSACHUSETTS

RESERVOIR Nº 2 DAM
MA 00338

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM MASS. 02154

JULY, 1981

UNCLASSIFIED

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19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

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Merrimack River Basin
Framingham, Massachusetts
Sudbury River

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Reservoir No. 2 Dam, also known as Brackett Reservoir Dam, is an earth embankment structure located in the south central portion of Framingham, Mass. The dam has a masonry core and is 1340 feet long,with a stuctural height of 26.5 ft.; the hydraulic height is 20.7 ft. The dam is considered to be in fair condition. It is classified as intermediate in size with a high hazard potential. It is recommended that the owner engag a qualified engineer to investigate the erosion next to the spillway.

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02254

REPLY TO
ATTENTION OF:
NEDED

AUG 26 1981

Honorable Edward J. King
Governor of the Commonwealth of
Massachusetts
State House
Boston, Massachusetts 02133

Dear Governor King:

Inclosed is a copy of the Reservoir No. 2 Dam (MA-00338) Phase I Inspection Report, prepared under the National Program for Inspection of Non-Federal Dams. This report is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. I approve the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is vitally important.

Copies of this report have been forwarded to the Department of Environmental Quality Engineering, and to the owner, Commonwealth of Massachusetts, Metropolitan District Commission (MDC), Boston, MA. Copies will be available to the public in thirty days.

I wish to thank you and the Department of Environmental Quality Engineering for your cooperation in this program.

Sincerely,

C. E. EDGAR, III
Colonel, Corps of Engineers
Commander and Division Engineer

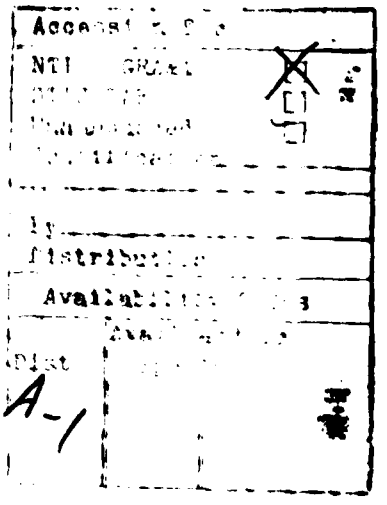
Incl
As stated

RESERVOIR NO. 2 DAM

MA 00338

MERRIMACK RIVER
FRAMINGHAM, MASSACHUSETTS

PHASE I - INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM



NATIONAL DAM INSPECTION PROGRAM
PHASE I - INSPECTION REPORT
BRIEF ASSESSMENT

Identification No.:	MA 00338
Name of Dam:	Reservoir No. 2
City:	Framingham
County and State:	Middlesex County, Massachusetts
Stream:	Sudbury River
Date of Inspection:	December 8, 1980

Reservoir No. 2 Dam, also known as Brackett Reservoir Dam, is an earth embankment structure located in the south-central portion of Framingham, Massachusetts, approximately one mile upstream of Reservoir No. 1 (Stearns Reservoir) Dam. The dam has a masonry core and is 1,340 feet long. The structural height is 26.5 feet; the hydraulic height is 20.7 feet. The overflow spillway is a stone masonry structure approximately 184.6 feet long. The spillway discharges directly into Reservoir No. 1.

The dam is owned and operated by the Metropolitan District Commission to impound water in Reservoir No. 2 and to regulate flow in the Sudbury River. The reservoir has a storage capacity of 2,800 acre-feet.

As a result of the visual inspection and a review of available data, Reservoir No. 2 Dam is considered to be in fair condition. Major concerns include seepage through the spillway, inadequate spillway capacity to pass the test flood discharge, significant erosion of the earth embankment at the interface of the spillway training wall, and lack of periodic maintenance of the embankment slope.

The dam is classified as intermediate in size and a high hazard structure in accordance with the recommended guidelines established by the Corps of Engineers. The test flood for this dam equals the Probable Maximum Flood (PMF). The test flood inflow was estimated to be 24,045 cubic feet per second (cfs) and resulted in an outflow discharge estimated to be 22,900 cfs. This would overtop the dam crest by about 1.7 feet. The maximum spillway capacity with the water level at top of dam was estimated to be 13,500 cfs, which is about 59 percent of the test flood discharge. A major breach to Reservoir No. 2 Dam would cause Reservoir No. 1 Dam approximately 0.5 mile downstream to be overtopped by approximately 4 feet. It is estimated that approximately 25 houses and three

industrial buildings would be subject to 5-8 feet of backwater flooding. It is estimated that Winter Street would be overtopped by about 3 feet, the Amtrak railroad bridge would be overtopped by about 1 foot, Franklin Street would be overtopped by about 3 feet, and Union Avenue would be overtopped by about 3 feet.

It is recommended that the owner engage a qualified registered professional engineer to investigate the cause of the erosion next to the spillway and the seepage through the spillway, and to perform a detailed hydrologic-hydraulic investigation to assess further the potential of overtopping the dam and the need for and the means to increase project discharge capacity. The owner should also repair the embankment erosion, repoint spillway joints, remove the abandoned walkway system, and establish a regular program for vegetation control. A visual inspection should be made once a month and a comprehensive technical investigation conducted once a year. A Surveillance program should be established for use during flood periods at the dam, and a downstream warning program developed.

The recommendations and remedial measures are described in Section 7 and should be addressed by the owner within one year after receipt of this Phase I Inspection Report.

Howard Shaevitz, P.E.
Project Manager
M.P.E. No. 28447

SCHOENFELD ASSOCIATES, INC.
Boston, Massachusetts

This Phase I Inspection Report on Reservoir No. 2 Dam (MA-00338) has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgement and practice, and is hereby submitted for approval.

ARAMAST MAHTESIAN, MEMBER
Geotechnical Engineering Branch
Engineering Division

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

JOSEPH W. FINEGAN, JR., CHAIRMAN
Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation and analysis involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition, and the downstream damage potential.

The Phase I Investigation does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings, and other items which may be needed to minimize trespassing and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

RESERVOIR NO. 2 DAM

TABLE OF CONTENTS

Section			Page
Brief Assessment			i
Review Board Page			iii
Preface			iv
Table of Contents			v
Overview Photo			viii
Location Map			ix
REPORT			
1.	PROJECT INFORMATION		1-1
	1.1	General	1-1
		a. Authority	1-1
		b. Purpose	1-1
	1.2	Description of Project	1-1
		a. Location	1-1
		b. Description of Dam and Appurtenances	1-2
		c. Size Classification	1-2
		d. Hazard Classification	1-2
		e. Ownership	1-2
		f. Operator	1-3
		g. Purpose of Dam	1-3
		h. Design and Construction History	1-3
		i. Normal Operation Procedures	1-3
	1.3	Pertinent Data	1-3
		a. Drainage Area	1-3
		b. Discharge at Dam Site	1-3
		c. Elevation	1-4
		d. Reservoir	1-4
		e. Storage	1-5
		f. Reservoir Surface	1-5

Section		Page
	g. Dam	1-5
	h. Diversion and Regulating Tunnel	1-6
	i. Spillway	1-6
	j. Regulating Outlets	1-6
2.	ENGINEERING DATA	2-1
	2.1 Design	2-1
	2.2 Construction	2-1
	2.3 Operation	2-1
	2.4 Evaluation	2-1
	a. Availability	2-1
	b. Adequacy	2-1
	c. Validity	2-1
3.	VISUAL INSPECTION	3-1
	3.1 Findings	3-1
	a. General	3-1
	b. Dam	3-1
	c. Appurtenant Structures	3-2
	d. Reservoir Area	3-2
	e. Downstream Channel	3-2
	3.2 Evaluation	3-2
4.	OPERATIONAL AND MAINTENANCE PROCEDURES	4-1
	4.1 Operational Procedures	4-1
	a. General	4-1
	b. Description of any Warning System in Effect	4-1
	4.2 Maintenance Procedures	4-1
	a. General	4-1
	b. Operating Facilities	4-1
	4.3 Evaluation	4-1

Section		Page
5.	EVALUATION OF HYDRAULIC/HYDROLOGIC FEATURES	5-1
	5.1 General	5-1
	5.2 Design Data	5-1
	5.3 Experience Data	5-1
	5.4 Test Flood Analysis	5-1
	5.5 Dam Failure Analysis	5-2
6.	EVALUATION OF STRUCTURAL STABILITY	6-1
	6.1 Visual Observations	6-1
	6.2 Design and Construction Data	6-2
	6.3 Post-Construction Changes	6-2
	6.4 Seismic Stability	6-2
7.	ASSESSMENT, RECOMMENDATIONS, AND REMEDIAL MEASURES	7-1
	7.1 Dam Assessment	7-1
	a. Condition	7-1
	b. Adequacy of Information	7-1
	c. Urgency	7-1
	7.2 Recommendations	7-2
	7.3 Remedial Measures	7-2
	a. Operation and Maintenance Procedures	7-2
	7.4 Alternatives	7-3

APPENDICES

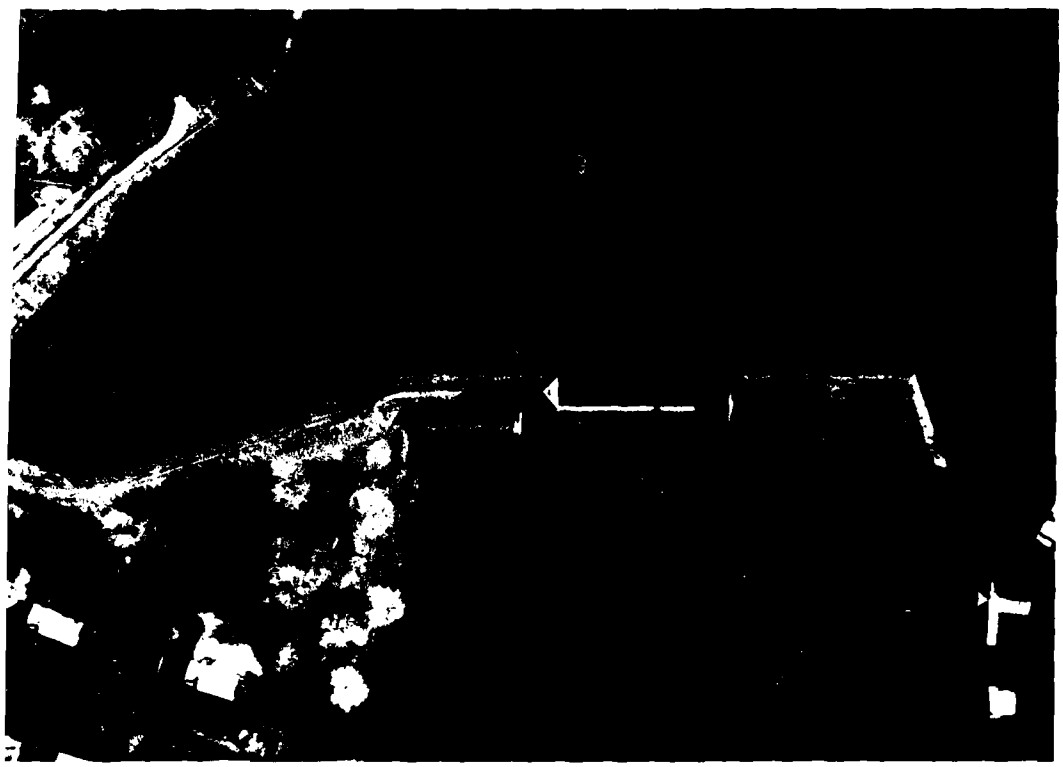
APPENDIX A - INSPECTION CHECK LIST

APPENDIX B - ENGINEERING DATA

APPENDIX C - SELECTED PHOTOGRAPHS

APPENDIX D - HYDROLOGIC AND HYDRAULIC COMPUTATIONS

APPENDIX E - INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS



OVERVIEW PHOTOGRAPHY
RESERVOIR NO. 2 (BRACKETT) DAM, FRAMINGHAM
RESERVOIR NO. 1 (STEARNS) IN FOREGROUND

22,900 cfs. This analysis indicated that the top of dam would be overtopped by approximately 1.7 feet. The maximum spillway capacity with the water level at the dam crest was estimated to be 13,500 cfs, which is 59 percent of the test flood discharge.

5.5 Dam Failure Analysis

The impact of dam failure with the reservoir surface at the dam crest was assessed utilizing the "Rule of Thumb" Guidance for Estimating Downstream Dam Failure Hydrographs provided by the Corps of Engineers. The analysis covered a reach extending approximately 10,000 feet downstream. A major breach to Reservoir No. 2 Dam would increase the stage of Reservoir No. 1 at its upper end by approximately 4 feet. Within the study area, an antecedent flow of 4,600 cfs was assumed. After a breach, the reach extending from the Amtrak Railroad bridge to Franklin Street would experience an increase in stage of about 4.4 feet, bringing the total depth along this reach to about 15 feet at the channel centerline. Approximately 15 houses along the north overbank would be subject to 5-8 feet of flooding. The reach extending from Franklin Street to Union Avenue would be subject to a similar increase and resultant total depth. About 10 houses along the north overbank would experience up to 7 feet of flooding. Also, an industrial complex on the south overbank would be flooded similarly. Based on this analysis, Reservoir No. 2 Dam was classified as a high hazard.

SECTION 5
EVALUATION OF HYDROLOGIC/HYDRAULIC FEATURES

5.1 General

Reservoir No. 2 Dam is an earth embankment structure with a masonry core. According to design drawings, the dam is 1,340 feet long and has a maximum structural height of 26.5 feet. The overflow spillway has a width of 17 feet and a length of 184.6 feet, and is located in the center of the site. The crest of the spillway is of stone-masonry construction. The spillway discharges directly into Reservoir No. 1.

The normal outlets consist of the two lowest 4.0-foot x 5.0-foot openings and are located underwater. The dam impounds Reservoir No. 2 which is part of the MDC's water supply system for the greater Boston area.

5.2 Design Data

No hydrological or hydraulic design data were disclosed.

5.3 Experience Data

Daily readings of the water surface elevations for the period of operation are maintained by the MDC. The records indicate that the highest surface elevation was 173.3 NGVD and occurred on January 19, 1979.

5.4 Test Flood Analysis

Due to the absence of detailed design and operational information, the hydrologic evaluation was performed utilizing data gathered during the field inspection, watershed size, and an estimated test flood equal to the Probable Maximum Flood (PMF). The full PMF test flood was selected because the dam is intermediate in size and is a high hazard. The drainage basin is essentially rolling; however, the coastal curve from the Corps of Engineers set of guide curves was used because the upper watershed includes Ashland, Hopkinton, and Whitehall Reservoirs as well as Cedar Swamp in Westborough.

Based on an estimated maximum probable flood peak flow rate of 545 cfs per square mile and a drainage area of 45.8 square mile, the test flood inflow was estimated to be 24,045 cfs. The test flood was routed through the dam in accordance with the Corps of Engineers procedure for Estimating Effect of Surcharge Storage on Maximum Probable Discharge. The reservoir water surface was assumed to be at elevation 171.6 NGVD prior to the flood routing. The project discharge was estimated to be

SECTION 4
OPERATIONAL AND MAINTENANCE PROCEDURES

4.1 Operational Procedures

a. General. Reservoir No. 2 Dam is part of the MDC's emergency water supply system on the Sudbury River. There are no formal operating procedures, although current practice is to allow continuous flow over the spillway in the summer to prevent neighborhood children from using the weir crest as a walkway.

b. Description of Any Warning System in Effect. No written warning system or emergency preparedness system exists for the dam.

4.2 Maintenance Procedures

a. General. The owner, the MDC, is responsible for maintenance of the dam. The site is visited daily. The grass on the crest of the dam is mowed regularly. The area downstream of the dam is also maintained regularly. There are no established procedures or manuals.

b. Operating Facilities. No formal maintenance procedures for the operating facilities were disclosed.

4.3 Evaluation

In general, the current operational and maintenance procedures appear inadequate to insure that normal problems can be remedied within a reasonable period of time. The dense growth of brush on the downstream slope between the spillway and left abutment and the minor growth of brush on the upstream slope of the entire embankment should be mowed and maintained.

The owner should also establish a surveillance program for use during flood periods at the dam. A downstream warning program to follow in case of emergency should also be developed.

Wheel tracks and inadequate vegetation on the crest of the embankment between the spillway and right abutment render the embankment susceptible to erosion and possible breaching should the dam be overtopped.

A dense growth of brush on the downstream slope of the embankment between the spillway and the left abutment and a minor growth of brush on the upstream slope of the entire embankment could eventually lead to seepage and piping problems if any of the brush reaches tree, falls over, and pulls out its roots, or if it dies and its roots rot. Also, as the brush becomes denser it will make it difficult to monitor the condition of the slope of the embankment.

Trees growing close to the downstream toe of the embankment between the spillway and left abutment could also cause seepage and piping problems if a tree falls over and pulls out its roots or if a tree dies or is cut and its roots rot.

Leakage of water through the joints in the downstream face of the stone-masonry spillway and freezing of water in the open joints from which the leakage is occurring will lead to long-term deterioration of the spillway structure.

Overall the general structural condition of the dam is fair. The visual inspection revealed items that lead to this assessment, such as:

(1) Seepage through the spillway.

(2) Significant erosion of the earth embankment at the spillway training wall.

(3) Lack of periodic maintenance of embankment slopes, specifically in the form of vegetation control.

Embankment section between spillway and left abutment - The crest of this portion of the embankment is covered with unmowed grass and weeds. Riprap on the upstream slope is in good condition and extends from a few feet below the crest to an undetermined elevation below the level of the reservoir at the time of the inspection. The downstream slope has a dense cover of weeds and brush. Severe erosion has occurred on the crest and downstream slope of the embankment next to the training wall at the left end of the spillway (Photo No. 9). A dense stand of pine trees is growing in the area immediately downstream of the toe of the embankment. No evidence of seepage from the embankment or downstream toe area was observed. The left abutment consists of soil and is in good condition.

c. Appurtenant Structures. There is a stone-masonry overflow-spillway structure in the central part of the dam. Some leakage is occurring through the joints of the stone-masonry in several locations along the lower portion of the spillway, as evidenced by icing on the downstream side at the time of the inspection (Photo Nos. 10 and 11). The downstream toe of the spillway is below the level of the water in Reservoir No. 1 and, as a result, it was not possible to determine if significant leakage was occurring through the foundation of the spillway. The alignment of the spillway was noted as good.

The gatehouse located on the northern end of the spillway is in good condition with the gates being reported operable by the owner. Inlet and outlet structures were underwater and their condition not known.

d. Reservoir Area. The area immediately adjacent to the reservoir is generally gently sloped and moderately vegetated with brush and trees. The shoreline shows no sign of sloughing or erosion. A rapid rise in the water level of the pond will not endanger life or property.

No evidence of significant sedimentation in the reservoir was observed.

e. Downstream Channel. As noted above, the water in Reservoir No. 1, which is located immediately downstream, backs up to the downstream toe of Reservoir No. 2 Dam.

3.2 Evaluation

On the basis of the visual inspection the dam is judged to be in fair condition. The crest, upstream slope, and downstream slope of the embankment next to the gatehouse is very severely eroded and, if not controlled, will result in breaching of the dam.

Very severe erosion of the crest and upstream slope of the embankment next to the left end of the overflow spillway structure, if not controlled, will also result in breaching of the dam.

SECTION 3
VISUAL INSPECTION

3.1 Findings

a. General. The visual inspection of Reservoir No. 2 Dam was conducted on December 8, 1980 by personnel from Schoenfeld Associates, Inc., Geotechnical Engineers, Inc., and D. Baugh & Associates, Inc. The inspection team was accompanied by Steven Kach of the Metropolitan District Commission. A copy of the visual inspection checklist completed during the field site visit is contained in Appendix A of this report. Selected photos of the dam are contained in Appendix C.

In general, the overall condition of the dam and its appurtenant structures is fair.

b. Dam. The dam is a masonry core structure consisting of an overflow stone masonry spillway with earthen embankments between the spillway and the abutments. Both the upstream and downstream faces are riprapped and appear to be in good condition (Photo Nos. 1 and 2). At the earth embankment/spillway training wall interfaces, significant erosion has occurred to the point where foundation stone is exposed. No seepage was noted through the dam.

Embankment section between spillway and right abutment - The crest of this portion of the embankment is not paved and has a sparse cover of grass which appears to have been mowed. There are wheel tracks on the crest (Photo No. 3). Riprap on the upstream slope extends from an elevation a few feet below the crest to an undetermined elevation below the level of the reservoir at the time of the inspection. The riprap is in good condition (Photo No. 4). Between the top of the riprap and the crest is a sparse cover of unmowed weeds and grass and a very small amount of brush. Unmowed grass and weeds cover the downstream slope, except that there is riprap on the bottom of the slope from a few feet above tailwater level to an undetermined elevation below tailwater level in the deeper section of the valley near the spillway. Next to the gatehouse, which is at the right end of the stone-masonry spillway, the crest, upstream slope (Photo No. 5), and downstream slope (Photo Nos. 6 and 7) are severely eroded. The area downstream of the embankment (Photo No. 8) is well maintained and shows no signs of seepage. The right abutment consists of soil and is in good condition. There is a row of large maple trees about 20 feet downstream of the toe of the dam, but they are not considered a potential problem because of the low height of the embankment and the distance from the toe (Photo No. 8).

SECTION 2
ENGINEERING DATA

2.1 Design

Design drawings showing a longitudinal section and several transverse sections were prepared for Reservoir No. 2 Dam in 1877 by the Metropolitan Water and Sewerage Board. The general design considerations are described in a Boston Water Works report dated 1882. The dam impounds one of three major water supply reservoirs on the Sudbury River.

2.2 Construction

The dam was constructed in 1878 by the Metropolitan Water and Sewerage Board. General construction features are described in a Boston Water Works report dated 1882.

2.3 Operation

Daily reservoir water surface elevations were the only operational records located during the investigation.

2.4 Evaluation

a. Availability. The engineering data used in the preparation of this report are presented in Appendix B.

b. Adequacy. In conjunction with the field inspection and computations, available engineering data and design drawings are considered adequate for a Phase I investigation.

c. Validity. The field investigation indicated that the external features of Reservoir No. 2 Dam have not changed substantially from the design drawings of 1877, except that the walkway across the spillway and the flashboards have been removed.

(7) Impervious core - granite

(8) Cutoff - none

(9) Grout curtain - none

(10) Other - none

h. Diversion and Regulating Tunnel - None

i. Spillway

(1) Type - stone-masonry

(2) Length of weir - 184.6 feet

(3) Crest elevation - 171.6 NGVD

(4) Gates - none

(5) U/S channel - not visible

(6) D/S channel - the downstream channel is the upper end of Reservoir No. 1

(7) General -

j. Regulating Outlets

(1) Inverts - two 4.0-foot x 5.0-foot openings at 153.7 one 48-inch opening at 153.7; two 4.0-foot x 5.0-foot openings at 163.7; one 4.0-foot x 4.5-foot opening at 167.7.

(2) Size - four openings 4.0 feet high and 5.0 feet wide; one opening 4.0 feet high and 4.5 feet wide; one opening 48 inches in diameter

(3) Description - the two lowest rectangular gates are used to control the water level in Reservoir No. 2

(4) Control mechanism - manually operated sluice gates located in gatehouse

(5) Other - none

(4) Design surcharge pool - 14,000

(5) Test flood pool - 14,530

(6) Top of dam - 14,000

e. Storage (gross acre-feet)

(1) Normal pool - 980

(2) Flood control pool - N/A

(3) Spillway crest pool - 980

(4) Design surcharge pool - 2,800

(5) Test flood pool - 3,280

(6) Top of dam - 2,800

f. Reservoir Surface (acres)

(1) Normal pool - 125

(2) Flood control pool - N/A

(3) Spillway crest pool - 145

(4) Design surcharge pool - 335

(5) Test flood pool - 380

(6) Top of dam - 335

g. Dam

(1) Type - gravel fill

(2) Length - 1,340 feet

(3) Hydraulic height - 20.7 feet

(4) Top width - 17 feet

(5) Side slopes - 10:7 H:V on upstream face; 10:6 H:V on downstream face

(6) Zoning - riprap on both slopes; granite block core wall; gravel fill

(2) Daily records of water surface elevation are maintained at the site. The maximum recorded elevation was 173.3 NGVD on January 25, 1979.

(3) The overflow spillway with the water surface at the top of the dam is approximately 13,500 cfs at elevation 179.5 NGVD.

(4) The overflow spillway with the water surface elevation at the test flood elevation of 181.2 NGVD is approximately 22,900 cfs.

(5) The total spillway capacity with the water surface at the test flood elevation of 181.2 is approximately 16,200 cfs.

(6) The total project discharge with the water surface at the top of the dam is approximately 13,500 cfs at elevation 179.5 NGVD.

(7) The total project discharge at the test flood elevation of 181.2 is approximately 22,900.

c. Elevation (feet above NGVD)

(1) Streambed at centerline of dam - 153.0

(2) Bottom of cutoff - N/A

(3) Maximum tailwater - 175.4 (test flood design surcharge in Section 1.3.C.(9) of the Phase I Dam Inspection Report for Reservoir No. 1)

(4) Normal pool - 171.7

(5) Flood control pool - N/A

(6) Spillway crest - 171.6

(7) Design surcharge - 173.7

(8) Test flood surcharge - 181.2

(9) Top of dam - 179.5

d. Reservoir (length in feet)

(1) Normal pool - 11,400

(2) Flood control pool - N/A

(3) Spillway crest pool - 11,400

f. Operator. The operation, maintenance, and safety of the dam is the responsibility of the Sudbury Section of the MDC, 133 Hollis Street, Framingham, Massachusetts. The Superintendent of the Sudbury Section is Mr. Edward Ginsburg (phone: (617) 872-4388).

g. Purpose of Dam. The dam was constructed to create a water supply reservoir for the metropolitan Boston area. The current purpose of the dam and reservoir is for an emergency water supply and to regulate flow on the Sudbury River. In the event the water is required for emergency purposes, it would flow from Reservoir No. 2 to Reservoir No. 1 to the Sudbury Aqueduct.

h. Design and Construction History. Reservoir No. 2 Dam was designed in 1877 and constructed in 1878 by the Metropolitan Water and Sewerage Board as one of three major water supply reservoirs on the Sudbury River. Copies of plans for the dam dated 1877 are located in Appendix B. Note that elevations shown on these plans are in feet above Boston City Base. This datum is 5.65 feet below NGVD.

i. Normal Operation Procedures. The dam is used for flow regulation into Reservoir No. 1 and as an emergency water supply by the MDC. Because of the reported poor quality of the water, it would only be used for drinking purposes during periods of severe emergency. Although there are no formal operating procedures, current practice is to allow continuous flow over the spillway in the summer to prevent use of the weir crest as a walkway by neighborhood children.

1.3 Pertinent Data

a. Drainage Area. The area tributary to Reservoir No. 2 Dam consists of 29,300 acres (45.8 square miles) of rolling terrain. There is a substantial amount of development in the watershed. Maximum watershed elevation is at about 700 feet(NGVD); full reservoir elevation is at 179.5 feet (NGVD).

The area around the reservoir is mostly wooded, with several houses along the shoreline. Reservoir No. 1 lies on the northwest side of the dam.

b. Discharge at Dam Site

(1) Outlet works for Reservoir No. 2 Dam consist of a stone masonry spillway, one circular opening and five rectangular openings. Maximum discharge of the openings when the reservoir is at the top of the dam (elevation 179.5 feet NGVD) is considered negligible because of the tailwater of Reservoir No. 1. The 184.6-foot long spillway has a crest at elevation 171.6 feet NGVD. When the water surface is at the top of the dam (elevation 179.5 feet NGVD), the spillway will have a capacity of 13,500 cfs.

b. Description of Dam and Appurtenances. Reservoir No. 2 Dam is an earth embankment structure with a masonry core. The dam is 1,340 feet long with a structural height of 26.5 feet and a hydraulic height of 20.7 feet. Both the upstream and downstream faces are riprapped. The dam impounds water in Reservoir No. 2. Reservoir No. 1 is immediately downstream of the dam. The area immediately adjacent to the reservoir is for the most part gently sloped and moderately covered with brush and trees.

The overflow spillway is a stone masonry structure approximately 184.6 feet long with a crest elevation of 171.6 feet (NGVD). There are training walls on either side, and there is a cast-iron framework of an abandoned walkway on the crest. The walkway was used to provide access to the spillway for the installation and removal of flashboards. Both the flashboards and the walkway have been removed.

A gatehouse is located on the right side of the spillway. The gates are still operable (see Appendix B for gate dimensions).

The dam is identified as Site SU-1721 by the Soil Conservation Service in its Inventory of Potential and Existing Upstream Reservoir Sites - Sudbury, Assabet and Concord Study Areas, Massachusetts.

c. Size Classification. The dam is considered to be intermediate in size because the hydraulic height is 20.7 feet and the storage is 2,800 acre-feet. This is in accordance with the Recommended Guidelines for Safety Inspections for Dams, which defines an intermediate dam as having a storage capacity of 1,000 to 50,000 acre-feet.

d. Hazard Classification. The potential for hazard posed by this dam is classified as high. This is in accordance with the Recommended Guidelines for Safety Inspection for Dams, which defines a high hazard structure as one which poses a threat to more than a few lives. A major breach to Reservoir No. 2 Dam would cause Reservoir No. 1 Dam approximately 0.5 mile downstream to be overtopped by approximately 4 feet. Approximately 25 houses and three industrial buildings would be subject to 5-8 feet of flooding along a reach extending from the Amtrak railroad bridge downstream to Union Avenue. Winter Street would be overtopped by about 3 feet, the Amtrak railroad bridge would be overtopped by about 1 foot, Franklin Street would be overtopped by about 3 feet, and Union Avenue would be overtopped by about 2 feet.

e. Ownership. The dam is owned by the Commonwealth of Massachusetts, Metropolitan District Commission (MDC), 20 Somerset Street, Boston, Massachusetts 02108. The original owner was the Metropolitan Water and Sewerage Board.

NATIONAL DAM INSPECTION PROGRAM
PHASE I - INSPECTION REPORT
RESERVOIR NO. 2 DAM

SECTION 1
PROJECT INFORMATION

1.1 General

a. Authority. Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Schoenfeld Associates, Inc. has been retained by the New England Division to inspect and report on selected dams in the Commonwealth of Massachusetts. Authorization and notice to proceed were issued to Schoenfeld Associates, Inc. under a letter of October 30, 1980 from Colonel William E. Hodgson, Jr., Deputy Division Engineer. Contract No. DACW33-81-C-0010 has been assigned by the Corps of Engineers for this work.

b. Purpose

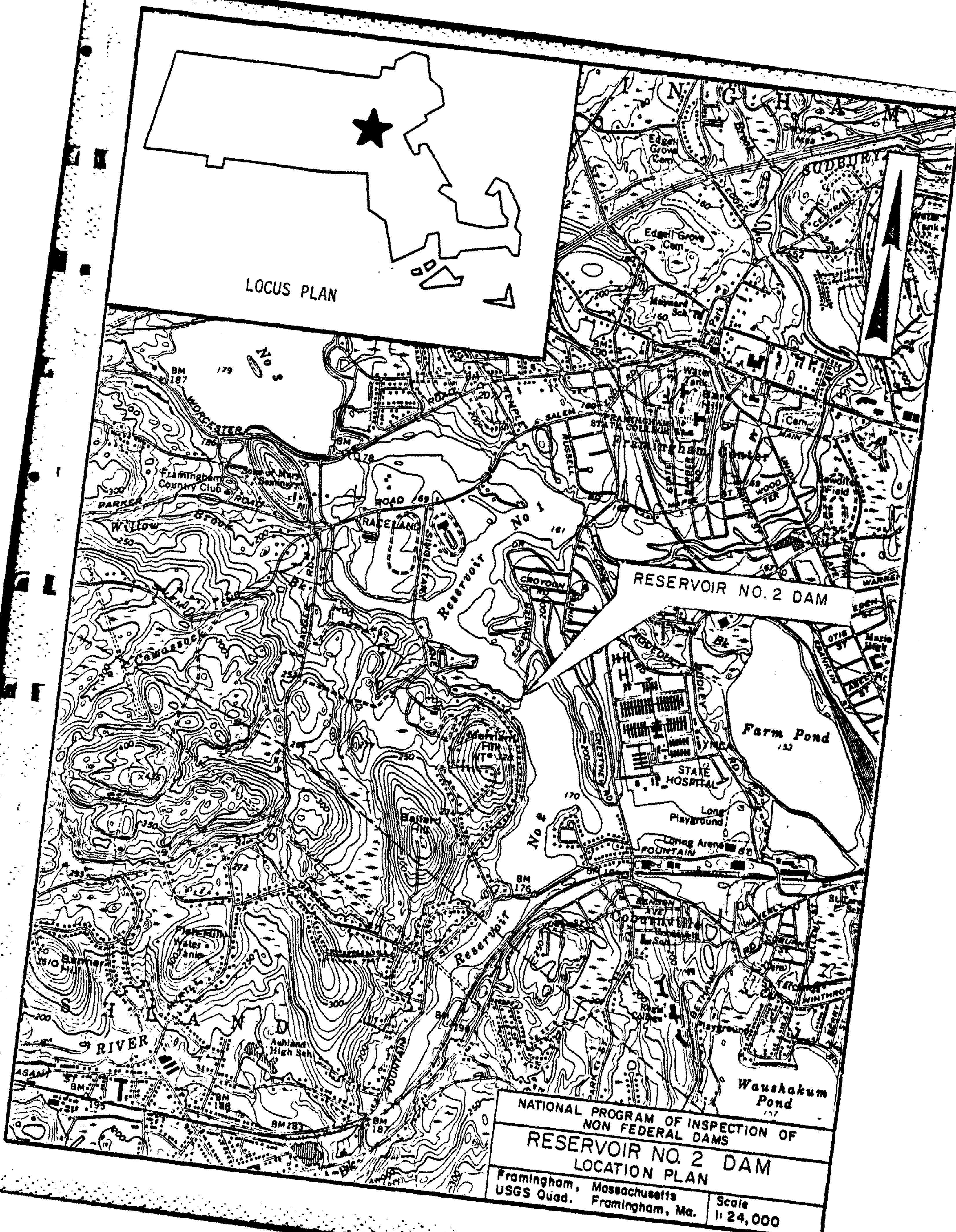
(1) To perform technical inspection and evaluation of nonfederal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by nonfederal interests.

(2) To encourage and prepare the states to initiate quickly effective dam safety programs for nonfederal dams.

(3) To update, verify, and complete the National Inventory of Dams.

1.2 Description of Project

a. Location. Reservoir No. 2 Dam, also known as Brackett Reservoir Dam, is located on the Sudbury River approximately 1.0 mile upstream of Reservoir No. 1 Dam and Winter Street in the town of Framingham, Massachusetts. The dam is shown on the U.S.G.S. quadrangle sheet of Framingham, Massachusetts. The approximate location is N-42°-17'-00' and W-71°-26'-42". The location of the dam is shown on the preceding page.


LOCUS PLAN
RESERVOIR NO. 2 DAM
Farm Pond
Framingham Center
STATE HOSPITAL
Waushakum Pond
Reservoir No 1
SUDBURY
Edgell Grove Cem
Framingham Country Club
RACELAND
Willow Brook
WORCESTER
PARKER ROAD
FOUNTAIN
Ashland High Sch
RIVER
NATIONAL PROGRAM OF INSPECTION OF NON FEDERAL DAMS
RESERVOIR NO. 2 DAM
LOCATION PLAN
Framingham, Massachusetts
USGS Quad. Framingham, Ma.
Scale 1:24,000

SECTION 6
EVALUATION OF STRUCTURAL STABILITY

6.1 Visual Observations

The general structural stability of the dam is good as evidenced by the vertical, horizontal, and lateral alignment. No seepage or any other form of distress was observed. The only areas of concern were the minimal seepage through the spillway and the earth erosion at the spillway training walls. However, neither is advanced to the point where the structural stability is in doubt.

The following conditions observed during the visual inspection, however, are indicative of problems that could result in long-term structural instability.

(1) Severe erosion of the crest, upstream slope, and downstream slope of the embankment exists next to the gatehouse.

(2) Severe erosion of the crest and upstream slope of the embankment exists next to the left end of the overflow spillway structure.

(3) Wheel tracks and inadequate vegetation on the crest of the embankment between the spillway and right abutment render the embankment susceptible to erosion.

(4) A dense growth of brush on the downstream slope of the embankment between the spillway and left abutment and a minor growth of brush on the upstream slope of the entire embankment could eventually lead to seepage and piping problems if any of the brush reaches tree size and blows over and pulls out its roots, or if it dies and its roots rot.

(5) Trees growing close to the downstream toe of the embankment between the spillway and left abutment could also cause seepage and piping problems.

(6) Leakage of water through the joints in the downstream face of the stone-masonry spillway and freezing of water in the open joints from which the leakage is occurring will lead to long-term deterioration of the spillway structure.

6.2 Design and Construction Data

Two drawings which show a longitudinal section and several transverse sections of the dam are available. They indicate that the stone-masonry spillway section of the dam is founded on ledge. The embankment sections of the dam appear to have a stone-masonry core wall about 5 feet thick which does not extend to bedrock and is underlain locally by compact sand and bounders, fine sand, coarse sand, and/or gravel. In the section of the embankment between the spillway and the right abutment, the stone-masonry core wall extends only about 300 feet from the spillway toward the right abutment, and a "puddle wall" appears to extend for the remaining 150 feet to the abutment. In a zone from about 25 to 85 feet left of the spillway, it appears that sheeting has been driven about 5 to 10 feet below the bottom of the stone-masonry core wall.

In general, it appears that the materials under the stone-masonry core wall of the embankment section of the dam may have a relatively high permeability and that some of them may be susceptible to piping. However, no evidence of seepage or piping was observed during the field inspection.

6.3 Post-Construction Changes

It appears that post-construction changes consist of the abandonment of the flashboard and walkway systems.

6.4 Seismic Stability

This dam is located in Seismic Zone No. 2, and in accordance with Corps of Engineers' guidelines does not warrant further seismic analysis at this time.

SECTION 7
ASSESSMENT, RECOMMENDATIONS, AND REMEDIAL MEASURES

7.1 Dam Assessment

a. Condition. Based on the results of the visual inspection, consideration of the available information, contact with the governing agency, and hydraulic/hydrologic computations, Reservoir No. 2 Dam is judged to be in fair condition. The following conditions may lead to long-term problems:

(1) Very severe erosion of the crest, upstream slope, and downstream slope of the embankment next to the gatehouse will result in breaching of the dam if not controlled.

(2) Very severe erosion of the crest and upstream slope of the embankment next to the left end of the overflow spillway structure will result in breaching of the dam if not controlled.

(3) Wheel tracks and inadequate vegetation on the crest of the embankment between the spillway and right abutment render the embankment susceptible to erosion and possible breaching if it should be overtopped.

(4) A dense growth of brush on the downstream slope of the embankment between the spillway and left abutment and a minor growth of brush on the upstream slope of the entire embankment could eventually lead to seepage and piping problems if any of the brush reaches tree size and blows over and pulls out its roots, or if it dies and its roots rot.

(5) Trees growing close to the downstream toe of the embankment between the spillway and left abutment could also cause seepage and piping problems if a tree blows over and pulls out its roots or if a tree dies or is cut and its roots rot.

(6) Leakage of water through the joints in the downstream face of the stone-masonry spillway and freezing of water in the open joints from which the leakage is occurring will lead to long-term deterioration of the spillway structure.

(7) The spillway is inadequate to carry the test flood discharge without the dam being overtopped.

b. Adequacy of Information. The information obtained from the design drawings and the results of the visual inspection are adequate for the purposes of this Phase I study.

c. Urgency. The owner should implement the recommendations in 7.2 and 7.3 within one year after receipt of this Phase I report.

7.2 Recommendations

The following investigations should be carried out and needed corrections performed under the direction of a registered professional engineer qualified in the design and construction of dams:

(1) Specify and oversee repairs for the erosion that has occurred on the embankment next to the gatehouse and next to the training wall at the left end of the overflow spillway structure.

(2) Specify and oversee the construction of erosion protection for the crest of the embankment between the gatehouse and the right abutment.

(3) Investigate seepage through the spillway as to its seriousness and solution.

(4) Specify and oversee procedures for removal of trees and their roots in a zone 25 feet wide at the downstream toe of the embankment between the spillway and the left abutment.

(5) Specify and oversee repairs to the stone-masonry spillway structure to prevent leakage between the stone blocks.

(6) Perform a detailed hydrologic-hydraulic investigation to assess further the potential of overtopping the dam and the need for and the means to increase project discharge capacity.

(7) Investigate the possible seepage in the foundation during periods of low water surface elevation in Reservoir No. 1. Deficiencies should receive immediate attention.

7.3 Remedial Measures

a. Operating and Maintenance Procedures. The owner should:

(1) Repair embankment erosion at both spillway abutments.

(2) Repoint spillway joints.

(3) Remove brush from the embankment and mow the embankment on a regular basis.

(4) Visually inspect the dam and appurtenant structures once a month.

(5) Inspect and operate all gates at least once a year to insure that they are in working condition. Deficiencies should receive immediate attention.

(6) Make a comprehensive technical inspection of the dam once every year under the direction of a registered engineer qualified in the design and construction of dams.

(7) Remove the supports for the abandoned walkway system flood periods at the dam.

(8) Establish a surveillance program for use during flood periods and also a downstream warning program to follow in case of emergency.

7.4 Alternatives

There are no practical alternatives to the above recommendations.

APPENDIX A

INSPECTION CHECK LIST

VISUAL INSPECTION CHECKLIST
PARTY ORGANIZATION

PROJECT Reservoir No. 2 Dam

DATE Dec. 8, 1980

TIME 1:00 P.M.

WEATHER Cloudy, Cool

W.S. ELEV. 164.8 BCB UPSTREAM
157.8 BCB DOWNSTREAM

PARTY:

1. Howard Shaevitz, SAI
2. Peter Austin, DBA
3. Ronald Hirschfeld, GEI
4. Steven Kach, MDC
5.
6.
7.
8.
9.
10.

	PROJECT FEATURE	INSPECTED BY	REMARKS
1.	Hydrology/Hydraulics	Howard Shaevitz	
2.	Structural Stability	Peter Austin	
3.	Soils and Geology	Ronald Hirschfeld	
4.			
5.			
6.			
7.			
8.			
9.			
10.			

PERIODIC INSPECTION CHECKLIST

PROJECT Reservoir No. 2 Dam DATE Dec. 8, 1980

PROJECT FEATURE Dam Embankment NAME

DISCIPLINE NAME

AREA EVALUATED	CONDITION
DAM EMBANKMENT	
Crest Elevation	173.83 BCB
Current Pool Elevation	164.8 BCB
Maximum Impoundment to Date	167.6 BCB on Jan. 19, 1979
Surface Cracks	None observed
Pavement Condition	Not paved
Movement or Settlement of Crest	None observed
Lateral Movement	None observed
Vertical Alignment	Good
Horizontal Alignment	Good
Condition at Abutment and at Concrete Structures	Severe erosion of embankment next to gatehouse & of downstream slope of embankment next to training wall at left end of spillway
Indications of Movement of Structural Items on Slopes	None observed
Trespassing on Slopes	No evidence of trespassing observed
Sloughing or Erosion of Slopes or Abutments	See "Condition of Abutment & at Concrete Structures"
Rock Slope Protection - Riprap Failures	Riprap on upstream & downstream slopes in good condition
Unusual Movement or Cracking at or Near Toe	None observed
Unusual Embankment or Downstream Seepage	None observed
Piping or Boils	None observed
Foundation Drainage Features	None observed
Toe Drains	None observed
Instrumentation System	None observed
Vegetation	Mowed grass on embankment between gatehouse & right abutment. Unmowed grass on crest, minor brush on upstream slope & heavy brush on downstream slope between spillway & left abutment

PERIODIC INSPECTION CHECKLIST

PROJECT Reservoir No. 2 Dam DATE Dec. 8, 1980

PROJECT FEATURE Dike Embankment NAME

DISCIPLINE NAME

AREA EVALUATED	CONDITION
DIKE EMBANKMENT	Not applicable
Crest Elevation	
Current Pool Elevation	
Maximum Impoundment to Date	
Surface Cracks	
Pavement Condition	
Movement or Settlement of Crest	
Lateral Movement	
Vertical Alignment	
Horizontal Alignment	
Condition at Abutment and at Concrete Structures	
Indications of Movement of Structural Items on Slopes	
Trespassing on Slopes	
Sloughing or Erosion of Slopes or Abutments	
Rock Slope Protection - Riprap Failures	
Unusual Movement or Cracking at or Near Toe	
Unusual Embankment or Downstream Seepage	
Piping or Boils	
Foundation Drainage Features	
Toe Drains	
Instrumentation System	
Vegetation	

PERIODIC INSPECTION CHECKLIST

PROJECT Reservoir No. 2 Dam DATE Dec. 8, 1980

PROJECT FEATURE Intake Channel NAME

DISCIPLINE NAME

AREA EVALUATED	CONDITION
OUTLET WORKS - INTAKE CHANNEL AND INTAKE STRUCTURE	
a. Approach Channel	
Slope Conditions	Good
Bottom Conditions	Not visible beneath reservoir pool
Rock Slides or Falls	None
Log Boom	None
Debris	None
Condition of Concrete Lining	Not applicable
Drains or Weep Holes	Not applicable
b. Intake Structure	
Condition of Concrete	Good (masonry)
Stop Logs and Slots	None

PERIODIC INSPECTION CHECKLIST

PROJECT Reservoir No. 2 Dam DATE Dec. 8, 1980

PROJECT FEATURE Control Tower NAME _____

DISCIPLINE _____ NAME _____

AREA EVALUATED	CONDITION
OUTLET WORKS - CONTROL TOWER	
a. Concrete and Structural	(masonry)
General Condition	Good
Condition of Joints	Good
Spalling	None
Visible Reinforcing	None
Rusting or Staining of Concrete	None
Any Seepage or Efflorescence	None observed
Joint Alignment	Good
Unusual Seepage or Leaks in Gate Chamber	None observed
Cracks	None
Rusting or Corrosion of Steel	Rust
b. Mechanical and Electrical	Not applicable
Air Vents	
Float Wells	
Crane Hoist	
Elevator	
Hydraulic System	
Service Gates	
Emergency Gates	
Lightning Protection System	
Emergency Power System	
Wiring and Lighting System	

PERIODIC INSPECTION CHECKLIST

PROJECT Reservoir No. 2 Dam DATE Dec. 8, 1980

PROJECT FEATURE Transition & Conduit NAME _____

DISCIPLINE _____ NAME _____

AREA EVALUATED	CONDITION
OUTLET WORKS - TRANSITION AND CONDUIT	(masonry)
General Condition of Concrete	Good
Rust or Staining on Concrete	None observed
Spalling	None observed
Erosion or Cavitation	None observed
Cracking	None observed
Alignment of Monoliths	Not applicable
Alignment of Joints	Good
Numbering of Monoliths	Not applicable

PERIODIC INSPECTION CHECKLIST

PROJECT Reservoir No. 2 Dam DATE Dec. 8, 1980

PROJECT FEATURE Outlet Structure NAME

DISCIPLINE NAME

AREA EVALUATED	CONDITION
OUTLET WORKS - OUTLET STRUCTURE AND OUTLET CHANNEL	(masonry)
General Condition of Concrete	Good
Rust or Staining on Concrete	None
Spalling	None
Erosion or Cavitation	None observed
Visible Reinforcing	None
Any Seepage or Efflorescence	None observed
Condition at Joints	Good
Drain Holes	None observed
Channel	Dam discharges directly into Reservoir No. 1
Loose Rock or Trees Overhanging Channel	None observed except for some trees on left side of channel
Condition of Discharge Channel	Good

PERIODIC INSPECTION CHECKLIST

PROJECT Reservoir No. 2 Dam DATE Dec. 8, 1980

PROJECT FEATURE Spillway Weir NAME

DISCIPLINE NAME

AREA EVALUATED	CONDITION
OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS	
a. Approach Channel	
General Condition	Good
Loose Rock Overhanging Channel	None observed
Trees Overhanging Channel	Some trees & brush overhang channel
Floor of Approach Channel	Soil
b. Weir and Training Walls	
General Condition of Concrete	Good, but some seepage noted through spillway
Rust or Staining	None
Spalling	None
Any Visible Reinforcing	None
Any Seepage or Efflorescence	None observed
Drain Holes	Drainholes in weir appear to be open. Drainholes in training wall are below tailwater
c. Discharge Channel	
General Condition	Good
Loose Rock Overhanging Channel	None
Trees Overhanging Channel	None
Floor of Channel	Not visible beneath tailwater in stilling basin
Other Obstructions	

Photo No. 7 - Severe erosion shown in Photo No. 6 viewed from crest.

Photo No. 8 - Downstream slope of dam viewed from gatehouse.

Photo No. 5 - Erosion of the crest and upstream slope at right side of gatehouse.

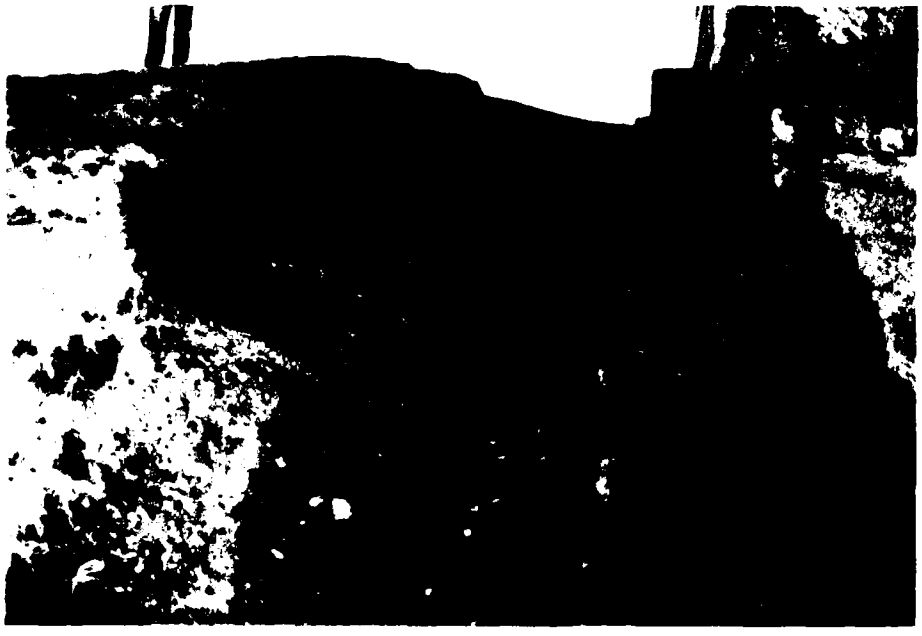


Photo No. 6 - Severe erosion of downstream slope of embankment on right side of gatehouse.

Photo No. 3 - Top of right embankment.

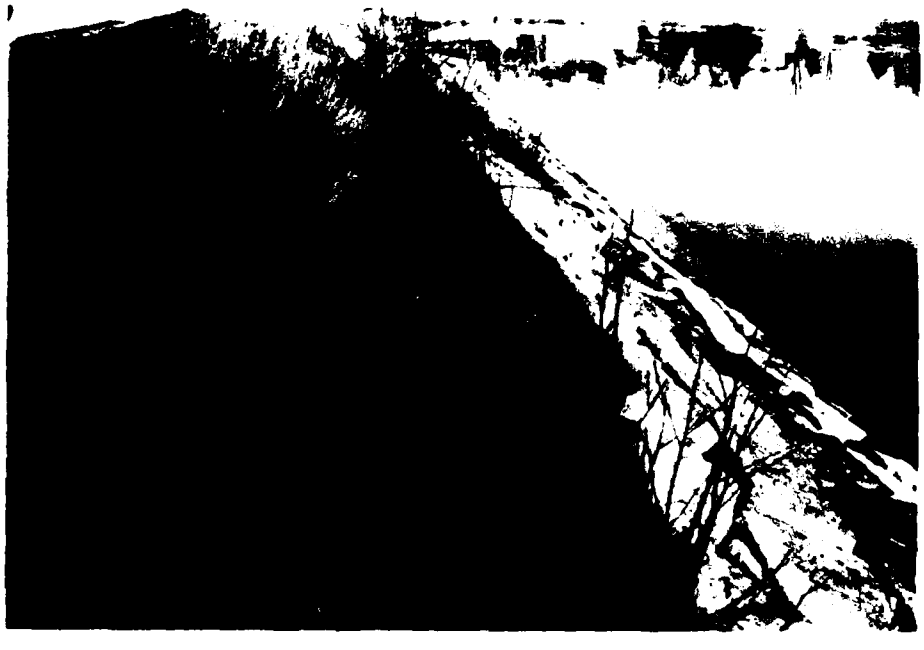


Photo No. 4 - Riprap on upstream slope to right of gatehouse.

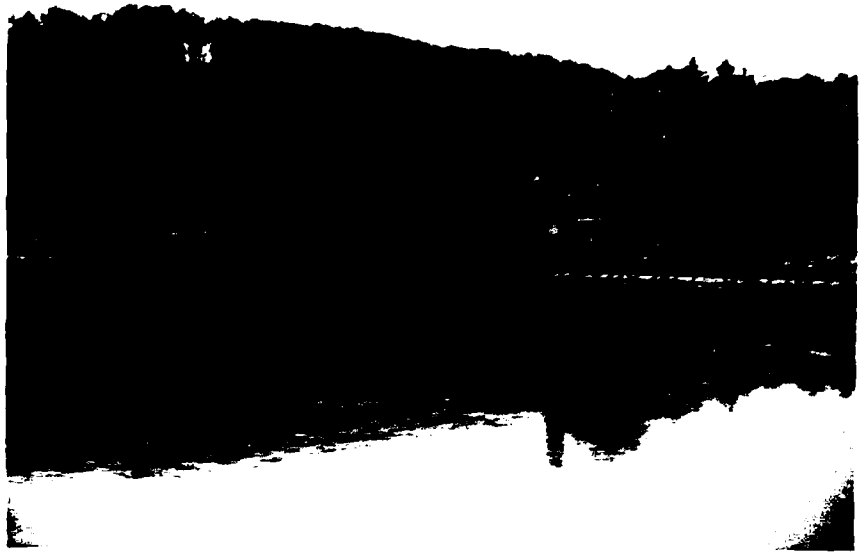


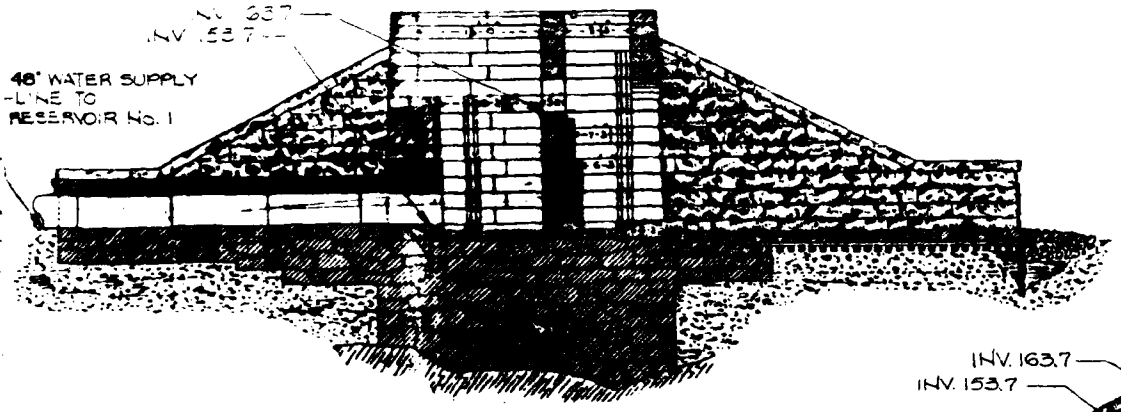
Photo No. 1 - Gatehouse and upstream faces of embankments.

Photo No. 2 - Gatehouse, spillway and downstream faces of embankments.

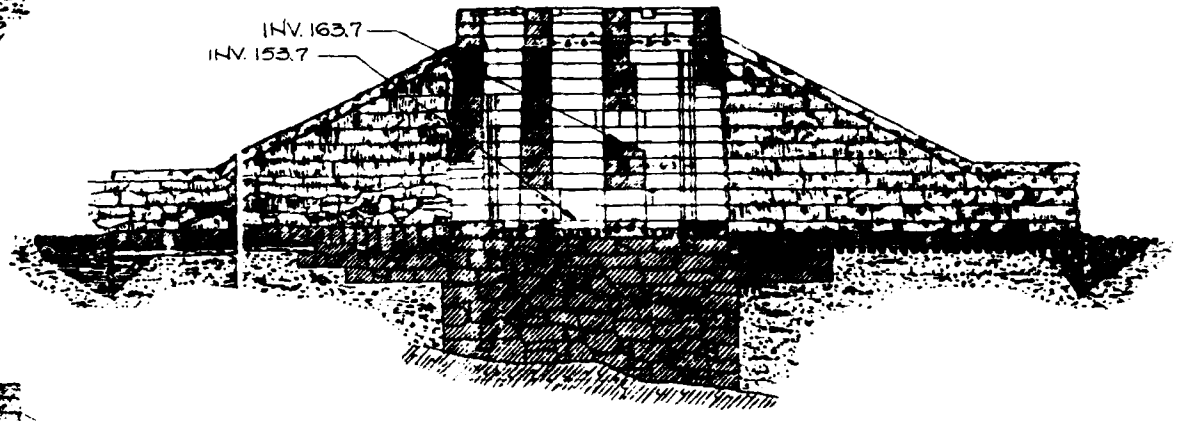
APPENDIX C

SELECTED PHOTOGRAPHS

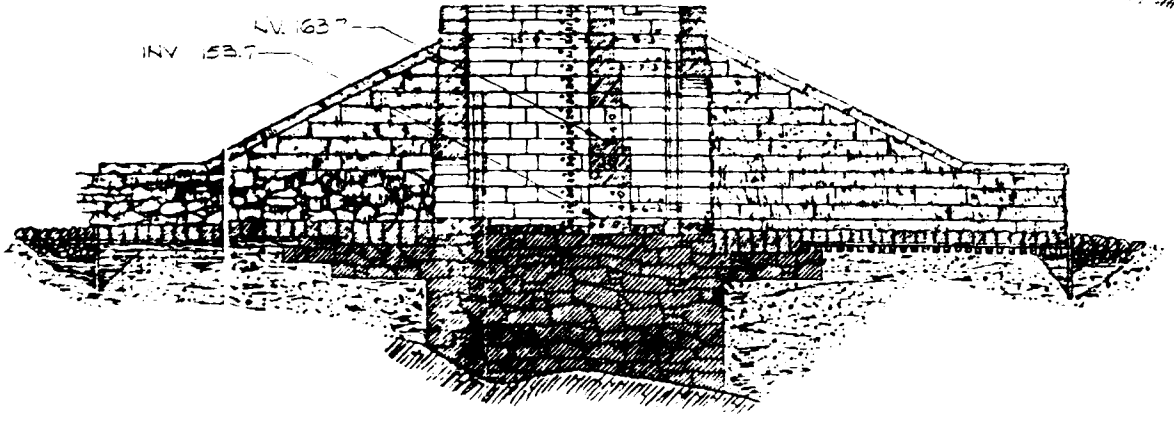
(The Index to these Photographs is found in Appendix B)



SECTION G-G



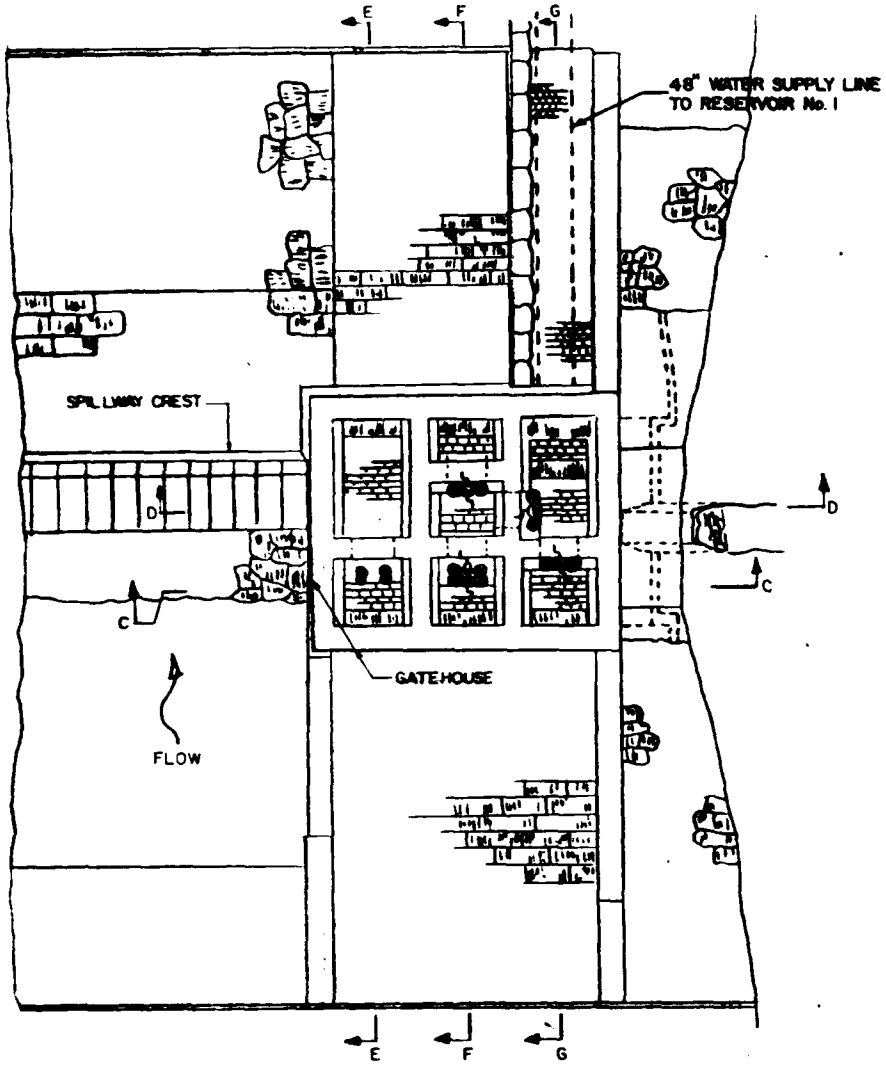
SECTION F-F



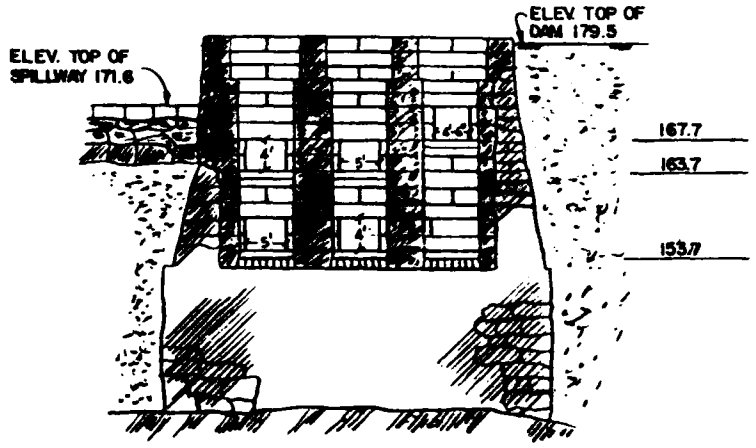
SECTION E-E

PLATE 6

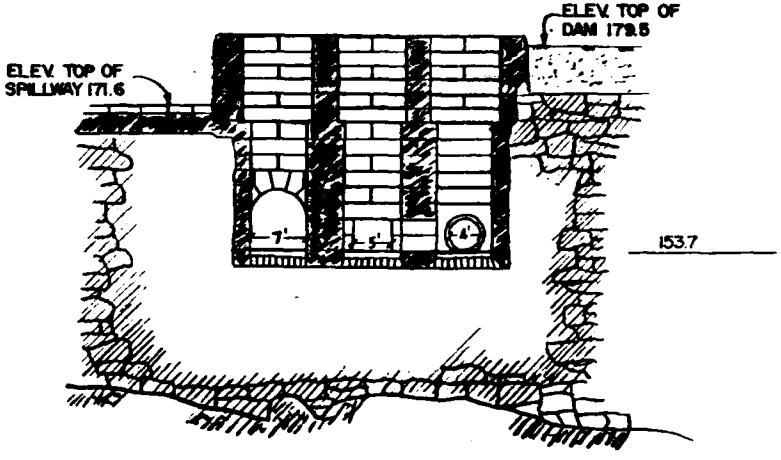
NATIONAL PROGRAM OF INSPECTION OF NON FEDERAL DAMS		
RESERVOIR NO 2 DAM SECTIONS		
Framingham,	Massachusetts	Scale



GATEHOUSE PLAN



SECTION C-C



SECTION D-D

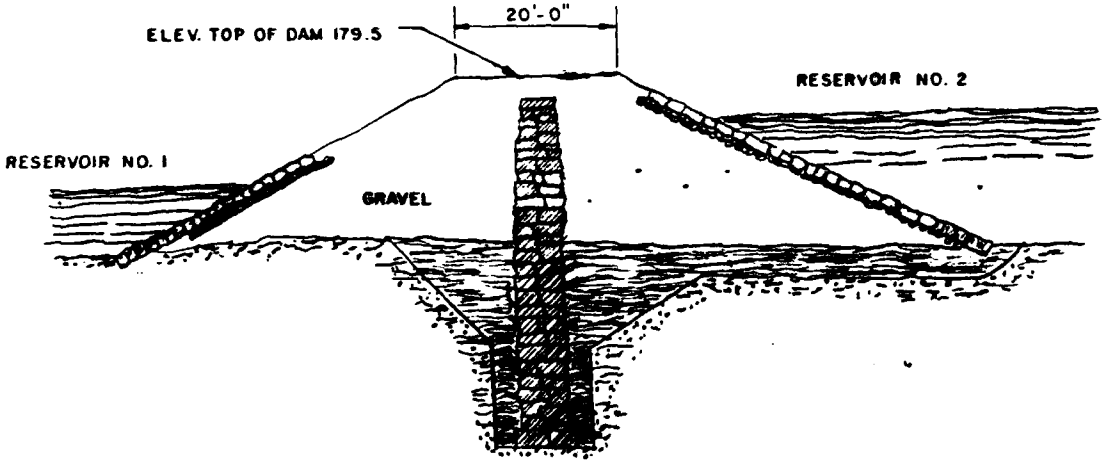
PLATE 5

NATIONAL PROGRAM OF INSPECTION OF NON FEDERAL DAMS

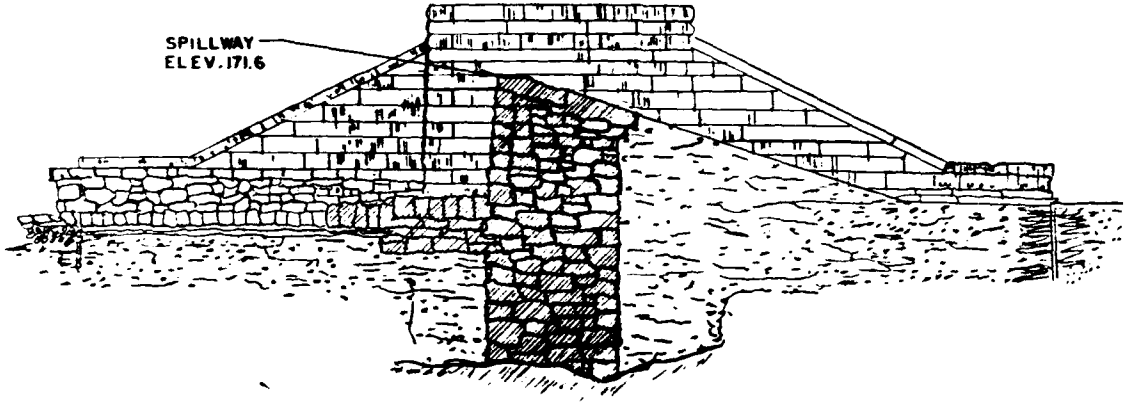
RESERVOIR NO. 2 DAM

Framingham, Mn

Scale



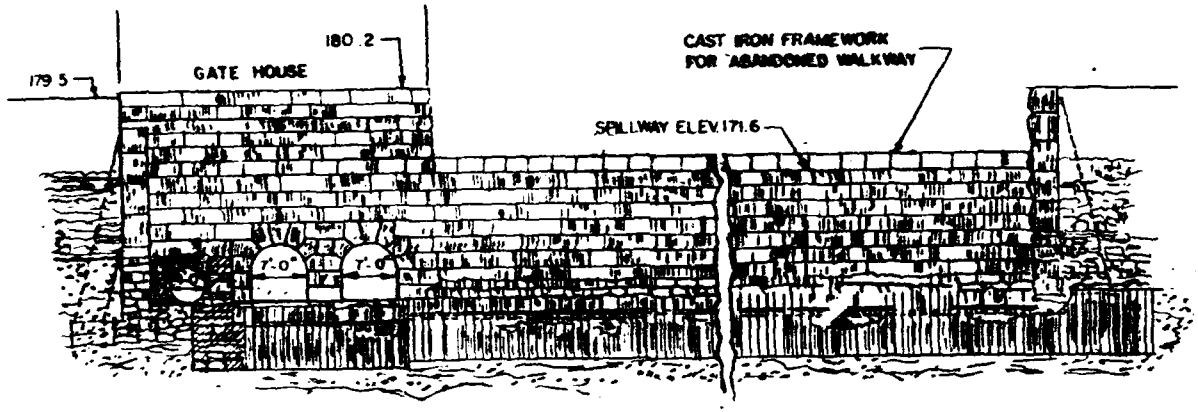
SECTION A-A



SECTION B-B

PLATE 4

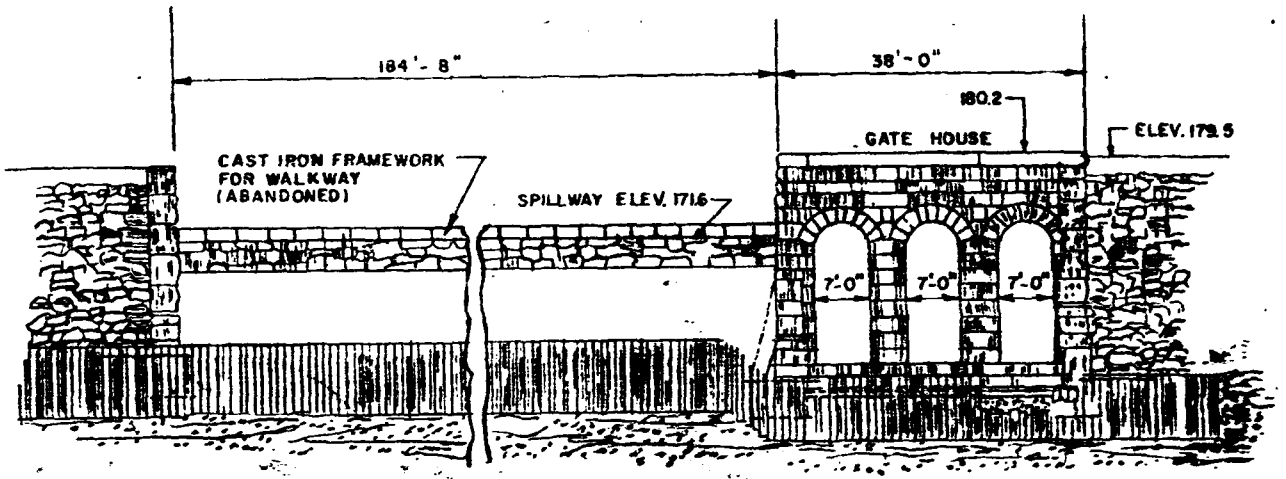
NATIONAL PROGRAM OF INSPECTION OF NON FEDERAL DAMS	
RESERVOIR NO. 2 DAM SECTIONS	
Framingham, Ma.	Scale



ELEVATION VIEW
DOWNSTREAM FACE

PLATE 3

NATIONAL PROGRAM OF INSPECTION OF NON FEDERAL DAMS
RESERVOIR NO. 2 DAM SECTIONS
Drawing No.	Scale



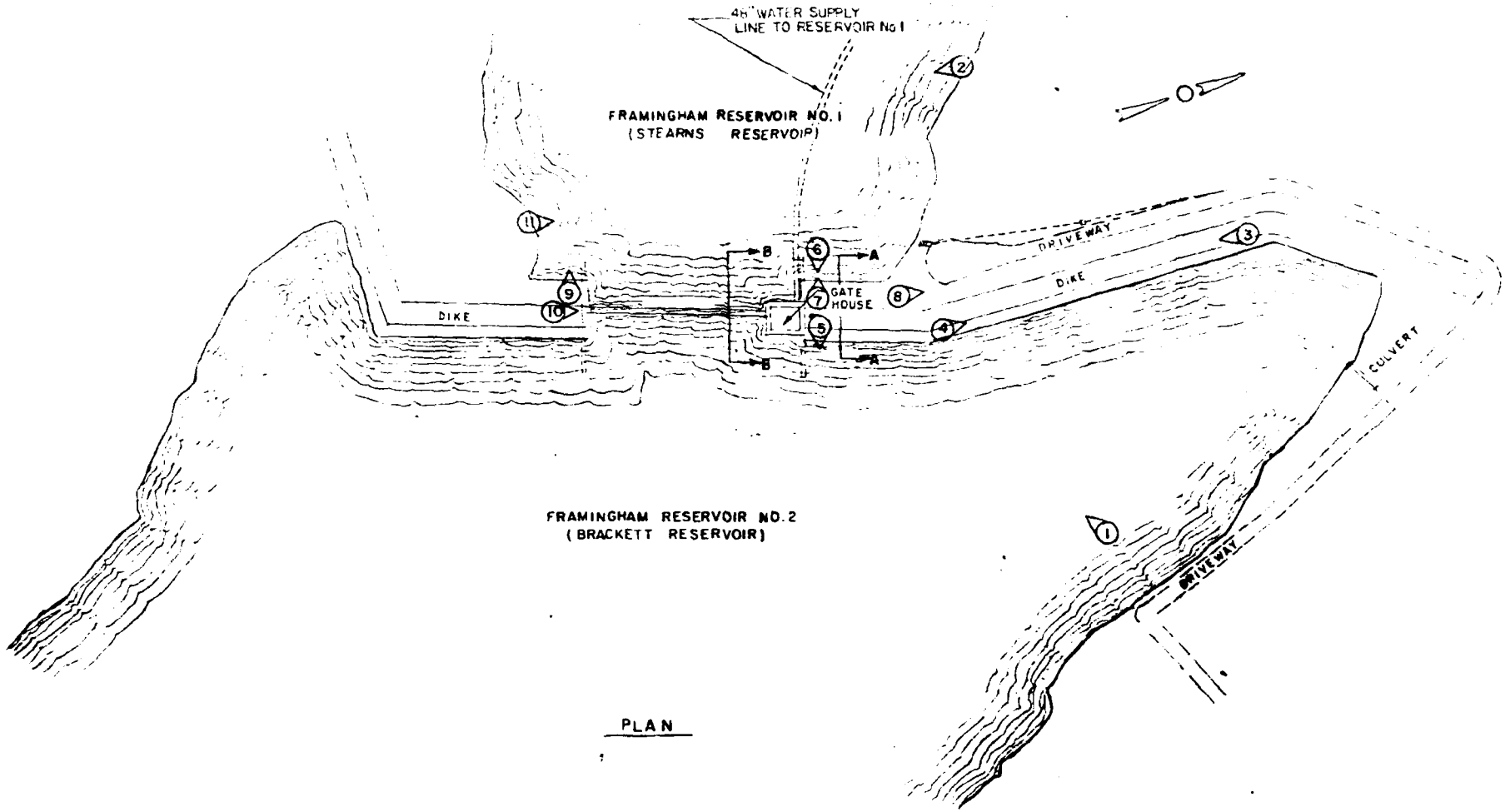
ELEVATION VIEW
UPSTREAM FACE.

PLATE 2

NATIONAL PROGRAM OF INSPECTION OF NON FEDERAL DAMS

RESERVOIR NO. 2 DAM
SECTIONS

Birmingham, Ala. | Scale



PLAN

(2) NUMBER AND ORIENTATION OF PHOTOGRAPHS IN APPENDIX C.

PLATE 1

NATIONAL PROGRAM OF INSPECTION OF NON FEDERAL DAMS

RESERVOIR NO [illegible] DAM

Framingham, Mass. | Scale

Available Engineering Data

Plans of the reservoir and dam were obtained from the MDC, Water Division, 20 Somerset Street, Boston, Massachusetts 02108. The drawings are dated 1877.

APPENDIX B

ENGINEERING DATA

PERIODIC INSPECTION CHECKLIST

PROJECT Reservoir No. 2 Dam DATE Dec. 8, 1980

PROJECT FEATURE Service Bridge NAME

DISCIPLINE NAME

AREA EVALUATED	CONDITION
OUTLET WORKS - SERVICE BRIDGE	Not applicable
a. Super Structure	
Bearings	
Anchor Bolts	
Bridge Seat	
Longitudinal Members	
Underside of Deck	
Secondary Bracing	
Deck	
Drainage System	
Railings	
Expansion Joints	
Paint	
b. Abutment & Piers	
General Condition of Concrete	
Alignment of Abutment	
Approach to Bridge	
Condition of Seat & Backwall	

Photo No. 9 - Severe erosion of downstream slope next to training wall at left end of spillway.

Photo No. 10 - Downstream side of granite spillway.

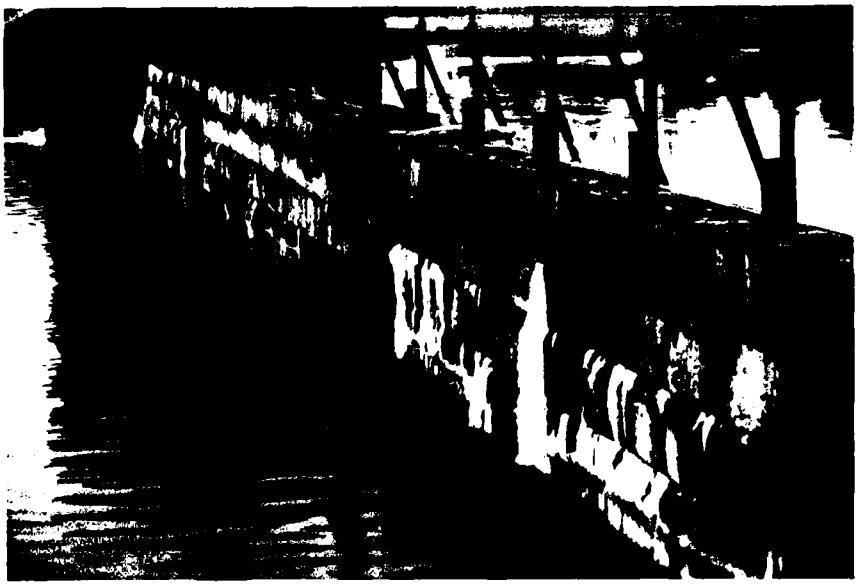
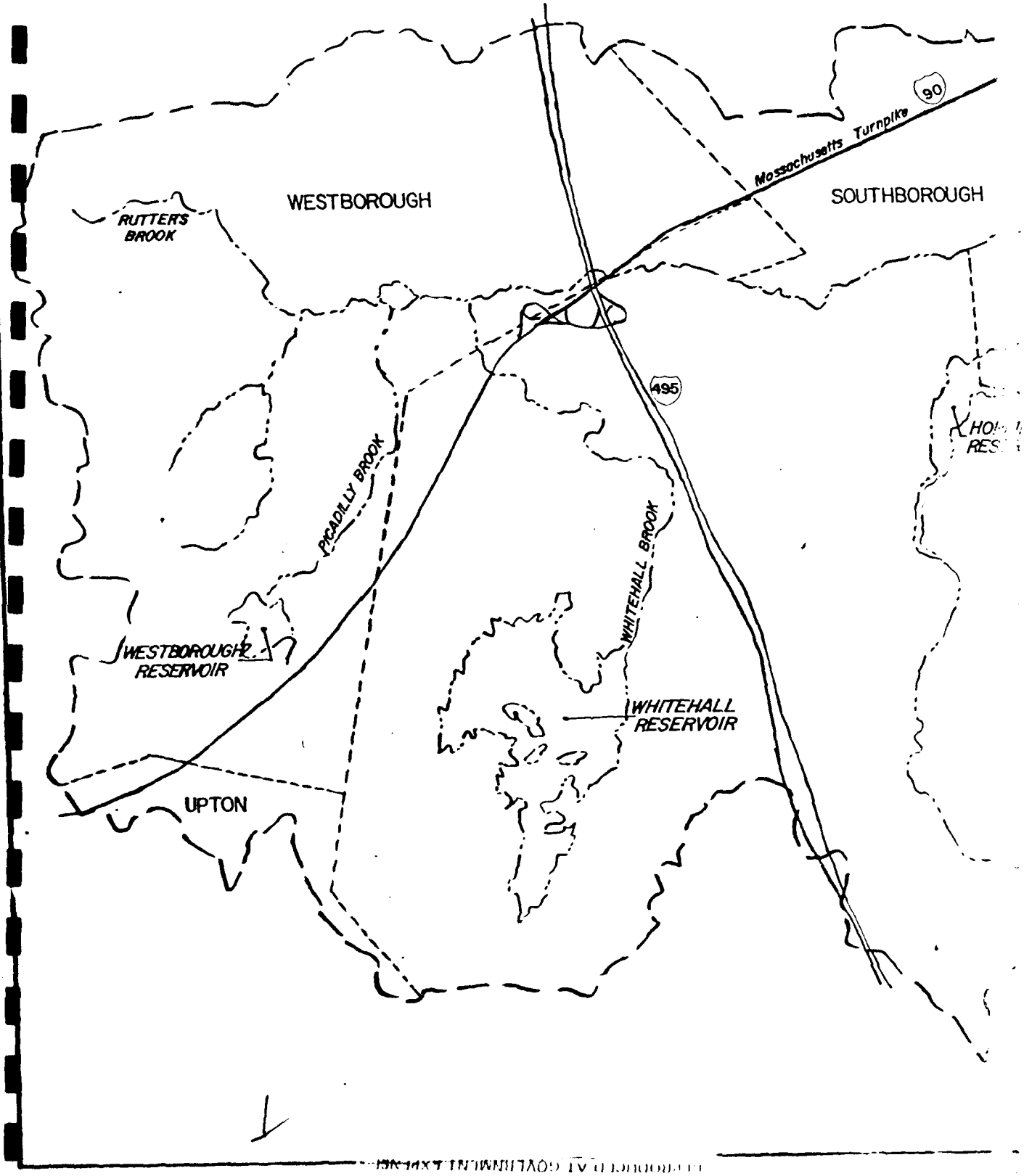
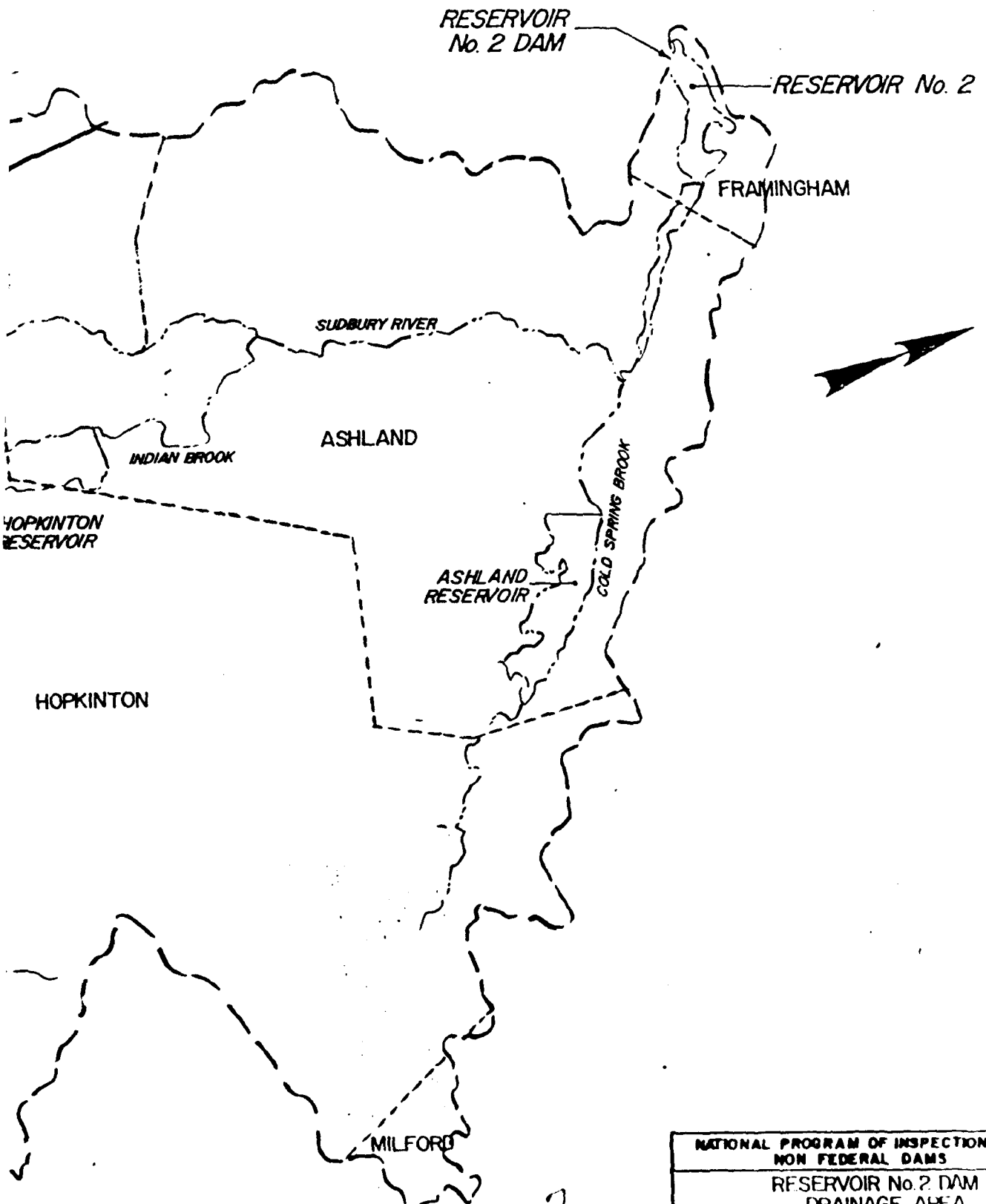


Photo No. 11 - Evidence of seepage through granite blocks on downstream side of spillway.

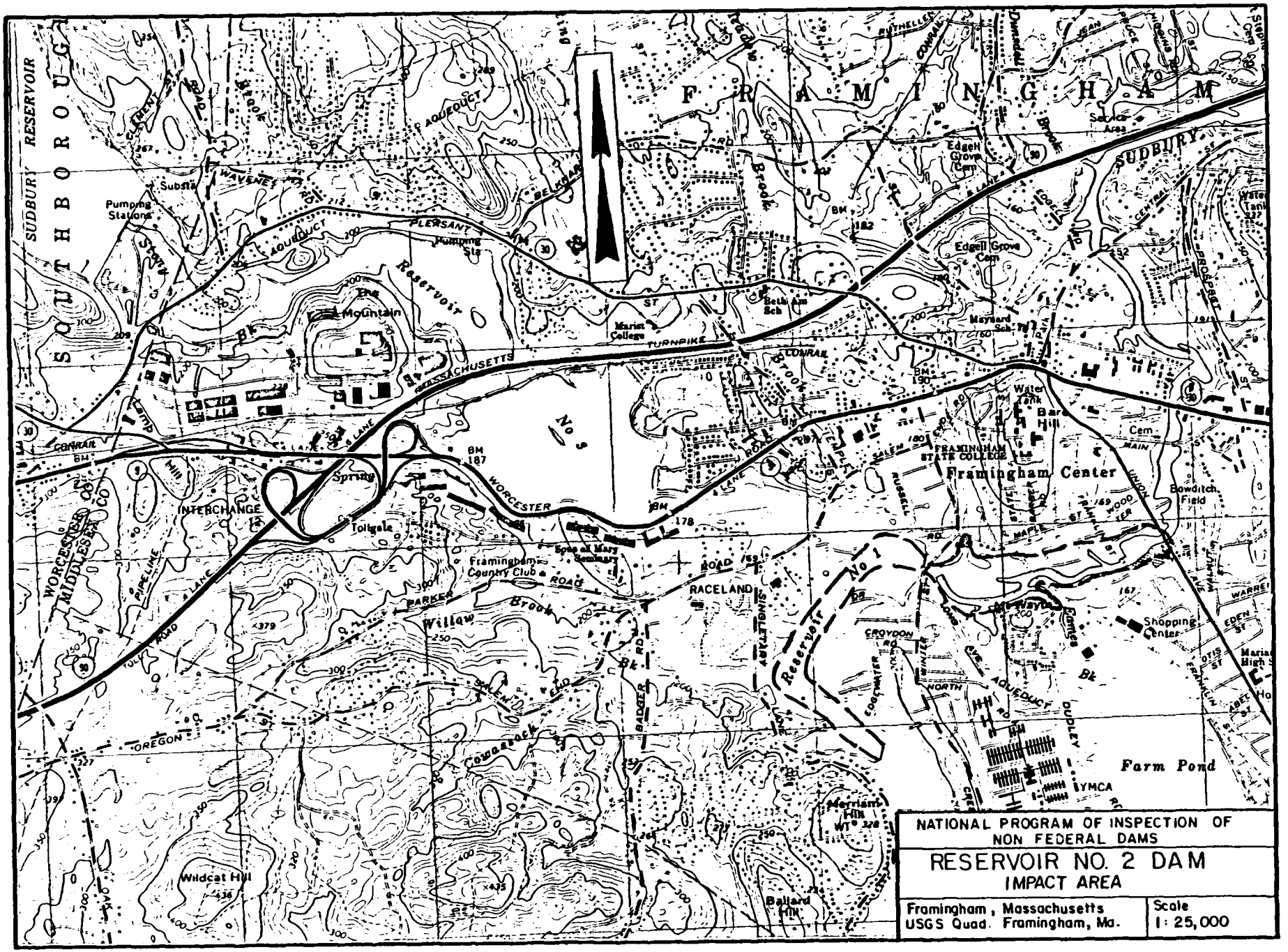
APPENDIX D

HYDROLOGIC AND HYDRAULIC COMPUTATIONS


WESTBOROUGH
SOUTHBOROUGH
RUTTERS BROOK
Massachusetts Turnpike
90
495
PICADILLY BROOK
WHITEHALL BROOK
WESTBOROUGH RESERVOIR
WHITEHALL RESERVOIR
UPTON



NATIONAL PROGRAM OF INSPECTION OF NON FEDERAL DAMS	
RESERVOIR No. 2 DAM DRAINAGE AREA	
Framingham, Massachusetts	Scale 1:48,000



NATIONAL PROGRAM OF INSPECTION OF NON FEDERAL DAMS

RESERVOIR NO. 2 DAM
IMPACT AREA

Framingham, Massachusetts USGS Quad. Framingham, Ma.	Scale 1: 25,000

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Consulting Engineers
210 South Street
BOSTON, MASSACHUSETTS 02111
(617) 423-5541

JOB FRAMINGHAM RES. NO. 2
SHEET NO. 1 OF 21
CALCULATED BY G. SHAREK DATE 3 APR 91
CHECKED BY H. Sheerin DATE Apr 24, 1991
SCALE

TEST FLOOD ANALYSIS

Choose spillway design flood (SDF)

Classification - Size: Intermediate
Hazard: High

Use probable maximum flood (PMF) as SDF

Upper watershed includes Ashland, Hopkinton, & Whitehall Reservoirs as well as Cedar Swamp in Westborough. Use guide curve for flat terrain...

$DA = 45.8\ mi^2$ $Q_p = 545\ csm$

$Q_{p_1} = 45.8\ mi^2 (545\ csm) \doteq 24945\ cfs$

Surcharge Storage Routing

$Q_{f_2} = Q_{p_1} - Q_{p_1}\left(\frac{STOR}{19}\right)$ * see note SH 2/21.

W.S. ELEV. ABOVE NGVD (FT)	SURCHARGE* STORAGE (AC-FT)	STOR (IN)	Q_{p_2} (CFS)
172	90	0.04	23994
174	400	0.16	23843
177	1100	0.45	23476
180	2000	0.82	23007
183	3200	1.31	22387
186	4700	1.92	21615

See surcharge storage routing curve, SH 4/21.

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JOB FRAMINGHAM RES. NO. 2
SHEET NO 2 OF 21
CALCULATED BY G. SHEEHY DATE 3 APR 81
CHECKED BY H. Shenton DATE Apr 24, 1981
SCALE

TEST FLOOD ANALYSIS.

*NOTE: Surcharge storage = total storage - storage at spillway crest of 980 ac-ft. See elevation vs. storage curves, SH 5/21.

Develop discharge rating curve at dam▽.

Granite overflow spillway is 185 feet long. Use $Q = CLH^{3/2}$ w/ C = 3.0 to account for walkway obstruction along crest.

Earth embankment is about 1150 feet long and 20 feet broad at top. Use C = 2.7.

Ignore insignificant flow through gate house openings.

W.S. ELEV. ABOVE NGVD (FT)	Q SPILLWAY (CFS)	Q EMBANKMENT (CFS)	Q TOTAL (CFS)
172	168		168
174	2128		2128
177	7061		7061
180	13633	1098	14731
181	16123	5704	21827
182	18749	12274	31023
183	21503	20331	41834
184	24381	29640	54021

See rating curve, SH 4/21.

▽ See SH 6/21 for weir elevation

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JOB FRAMINGHAM RES. NO. 2
SHEET NO. 3 OF 21
CALCULATED BY G. SHARZY DATE 3 APR 81
CHECKED BY L. Sheerin DATE April 24, 1981
SCALE

TEST FLOOD ANALYSIS

From the intersection of the discharge rating and surcharge storage routing curves, (SH 4/21) we have:

Test flood elevation ≐ 181.2 NGVD

Test flood outflow ≐ 22900 cfs

The dam embankment would be overtopped by about 1.7 feet during the test flood event.

Overtopping would also occur along a 1000-foot stretch of Winter St. just south of the State Hospital at the eastern shore of the reservoir. Depths of about 2.5 feet would be expected on Winter St. as water flowed east across Long Playground and into Farm Pond.

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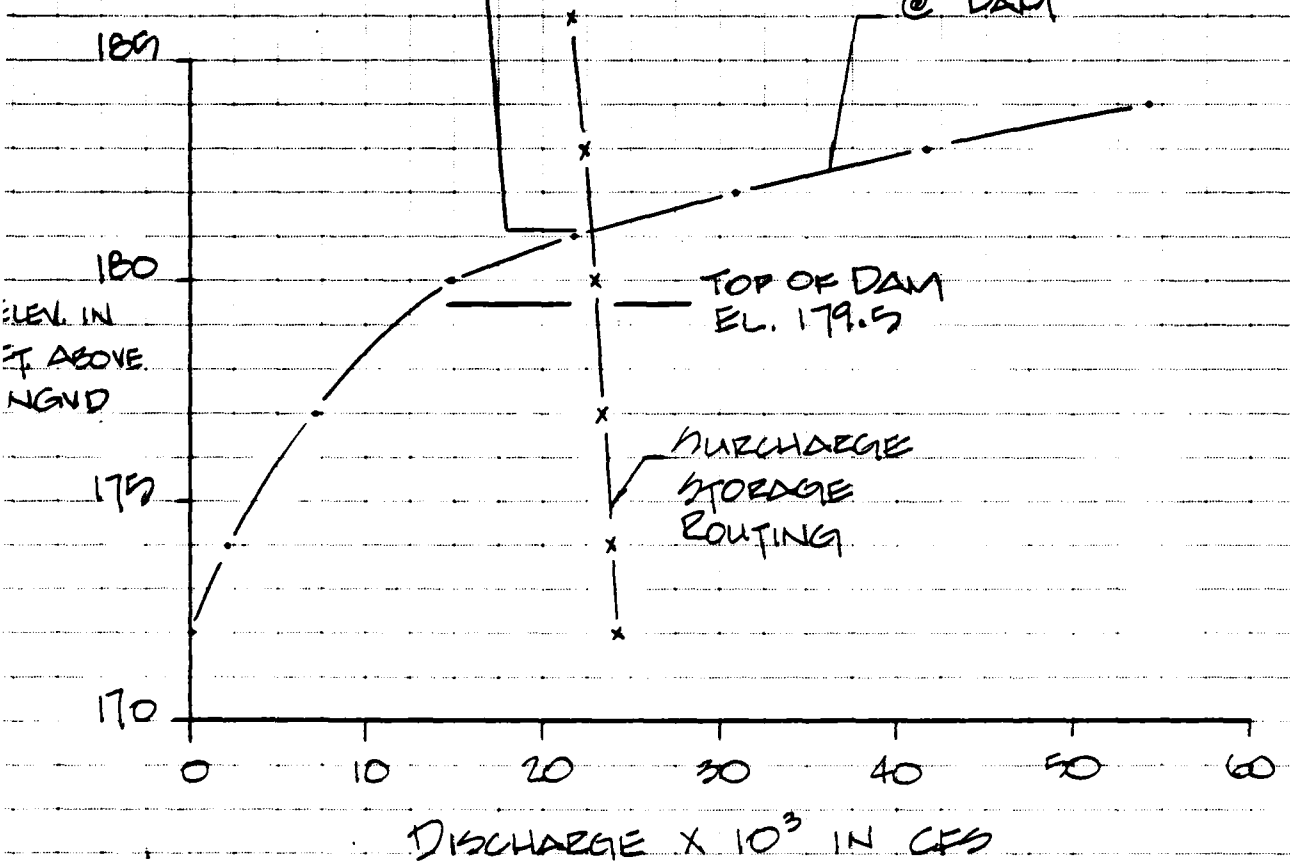
JOB FRAMINGHAM RES. NO. 2
SHEET NO. 4 OF 21
CALCULATED BY G. SHARRY DATE 2 APR 81
CHECKED BY H Shevitz DATE April 24, 1981
SCALE

ELEVATION VS. DISCHARGE

CURVE INTERSECTION

Q = 22900 CFS
ELEV = 181.2



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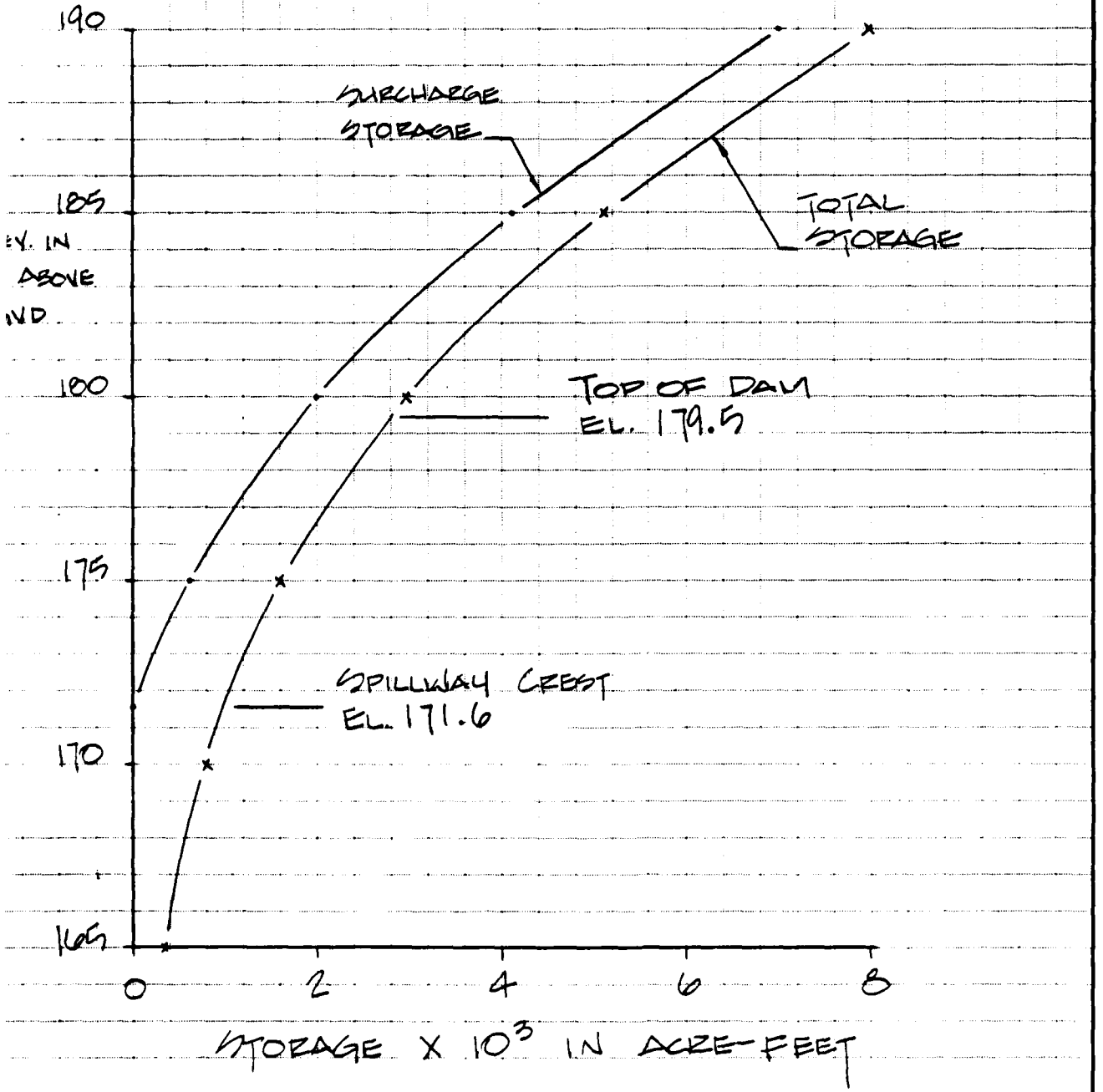
JOB FRAMINGHAM RES. NO. 2
SHEET NO. 5 OF 21
CALCULATED BY G. SHARRY DATE 2 APR 81
CHECKED BY H. Shevitz DATE April 14, 1981
SCALE

ELEVATION VS. STORAGE



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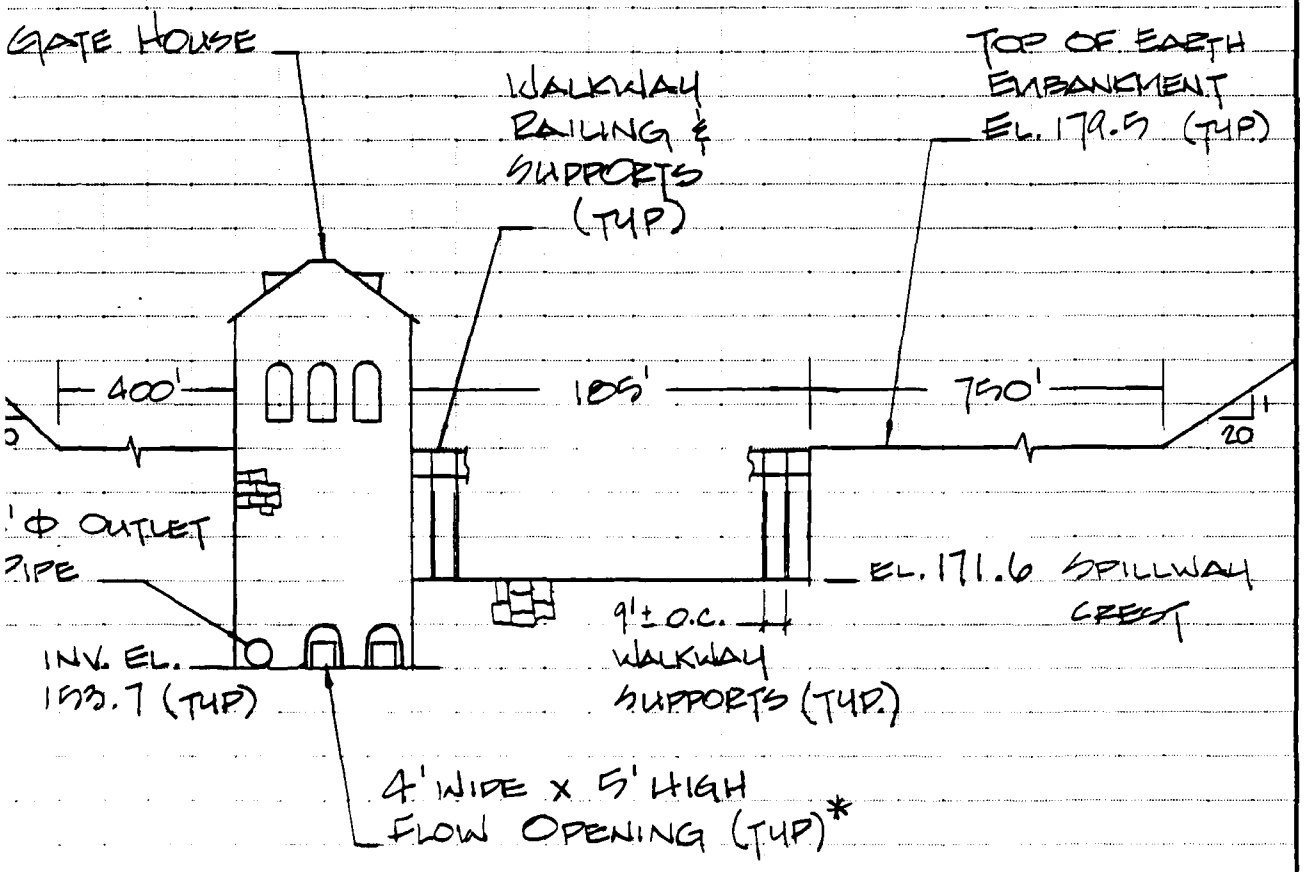
JOB FRAMINGHAM RES. NO. 2
SHEET NO. 16 OF 21
CALCULATED BY G. SHABEL DATE 3APR91
CHECKED BY L. Shrentz DATE April 29, 1991
SCALE

WEIR ELEVATION

NOTE: Elevations are in feet above NGVD.



LOOKING UPSTREAM

NOTE: See plans by Boston Water Works for "Dam No. 2" dated August, 1877 for additional detail on gate house flow scheme. (Appendix B)

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JOB FRAMINGHAM RES. NO. 2
SHEET NO. 7 OF 21
CALCULATED BY G. SHARRY DATE 7 APR 81
CHECKED BY M. Schenfeld DATE April 27, 1981
SCALE

BREACH ANALYSIS

Antecedant Condition

Assume spillway at Reservoir No. 1 downstream is at capacity with water surface elevation at 171.4 NGVD and discharge of 4600* cfs. This condition produces a tailwater with water surface about 0.15 feet below the crest of the spillway at Reservoir No. 2 prior to breach.

Compute breach outflow at Reservoir No. 2 ...

$Q_{P_1} = 8/27\ W_b \sqrt{g}\ H_o^{3/2}$

$W_b = 300$ ft. Use $H_o = 16$ ft.

$Q_{P_1} = 8/27\ (300)\ \sqrt{32.2}\ (16)^{3/2} = 32282$ cfs

Also, assume dam breaches along the earth embankment section. The spillway at Reservoir No. 2 would contribute a significant amount of outflow during a breach.

Spillway outflow $= CLH^{3/2}$, $C = 3.0$, $L = 185$ ft.
$H = 179.5 - 171.6 = 7.9$ ft.

$Q = 3(185)(7.9)^{3/2} = 12323$ cfs

Total Breach Outflow = 11413 + 12323 = 44605 cfs

See rating curve for "REACH 1", SH 20/21.

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JOB: FRAMINGHAM RES. No. 2
SHEET NO: 8 OF 21
CALCULATED BY: G. SHARPY DATE: [illegible]
CHECKED BY: H. Shmutz DATE: [illegible]
SCALE:

[illegible] ANALYSIS

REACH 1

[illegible] limit is Reservoir No. 1 Dam.
[illegible] rating curve for dam on p. D-4, COE [illegible] Report No. MA00337, as rating curve "REACH 1."

[illegible] Q @ Res. No. 1 dam = 4600 cfs
[illegible] stage = el. 171.4 NGVD.

[illegible] breach Q + antecedent Q
= 44605 + 4600 = 49205 cfs

[illegible]

[illegible] Res. No. 2
= [illegible] (top of dam) − [illegible] 171.4 (d/s water surface)

= [illegible] − [illegible] = 1786 ac-ft

[illegible] .82.7 [illegible]

[illegible]

APPENDIX E

INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS

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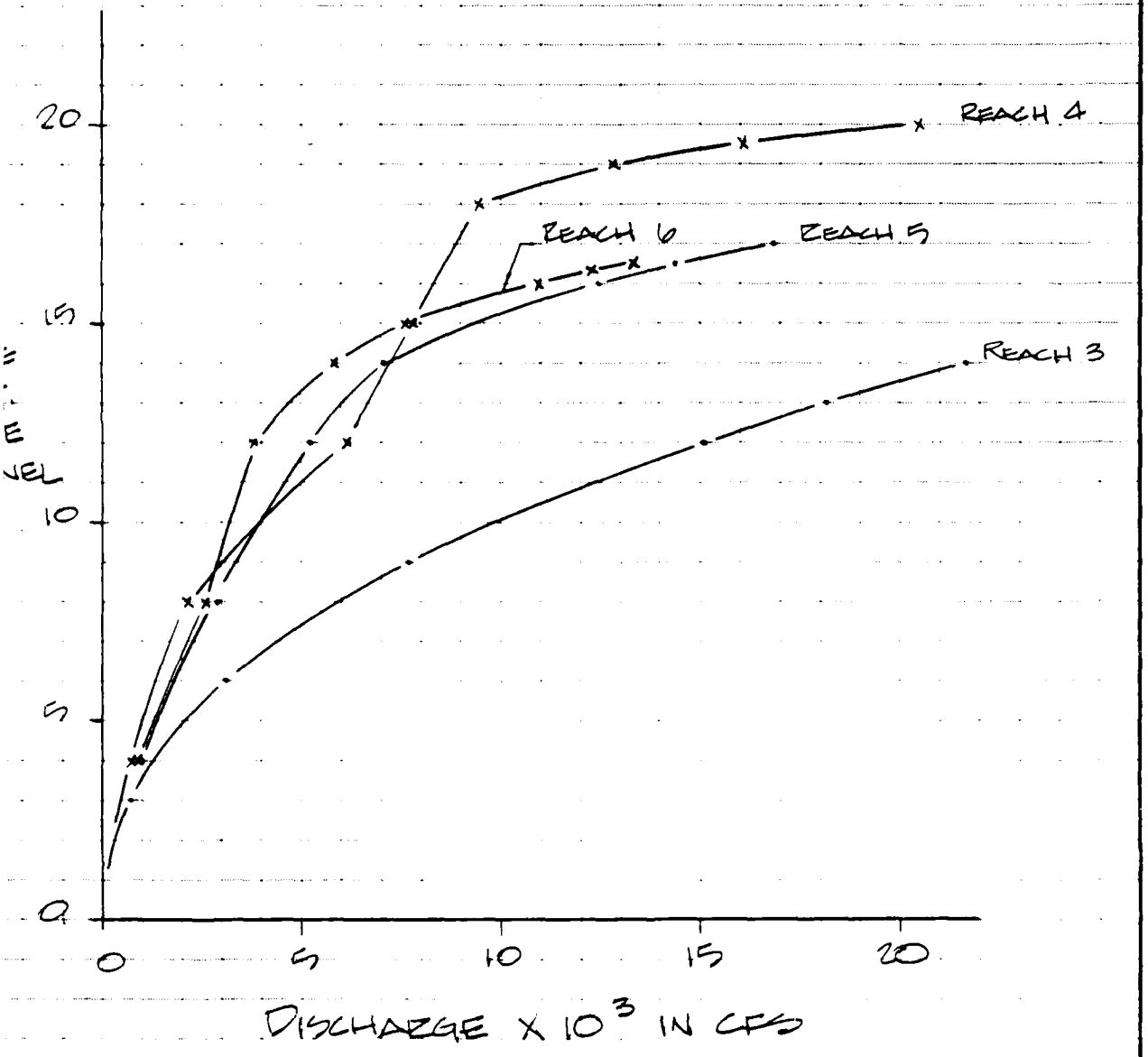
JOB FRAMINGHAM RES. NO. 2
SHEET NO. 21 OF 21
CALCULATED BY G. SHARRY DATE 6APR81
CHECKED BY H. Sheerin DATE April 27, 1981
SCALE

XCH ANALYSIS

DOWNSTREAM HAZARD - REACH RATING CURVES



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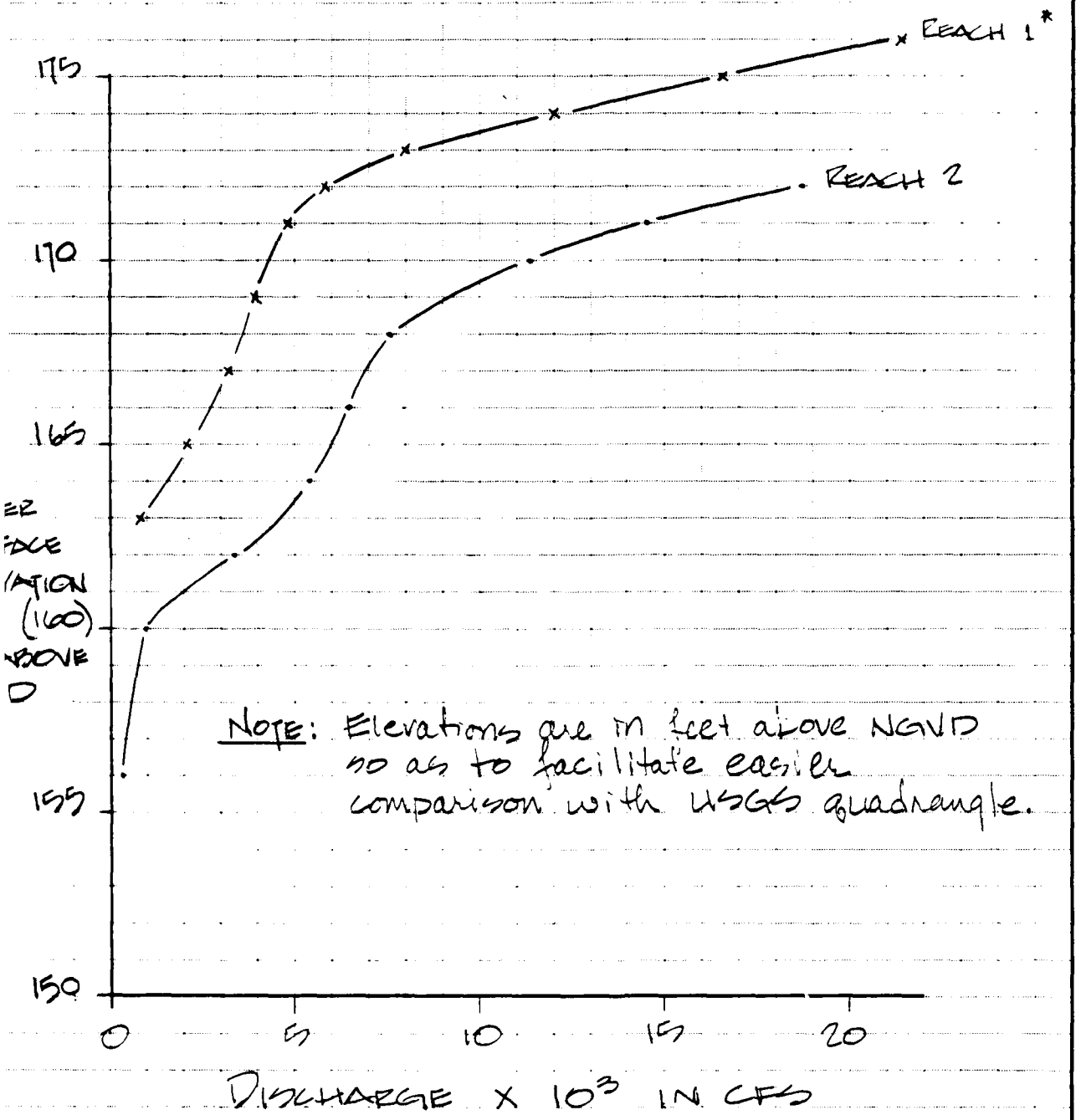
JOB FRAMINGHAM RES. NO. 2
SHEET NO. 20 OF 21
CALCULATED BY G. SHARRY DATE 6 APR 81
CHECKED BY H. Shaw Fz DATE April 27, 1981
SCALE

ACH ANALYSIS

DOWNSTREAM HAZARD - REACH RATING CURVES



NOTE: Elevations are in feet above NGVD so as to facilitate easier comparison with USGS quadrangle.

From COE Report No. MA 00337, Reservoir No. 1, p. D-4

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JOB FRAMINGHAM RES. NO. 2
SHEET NO. 19 OF 21
CALCULATED BY G. SHARRY DATE 9 APR 81
CHECKED BY H. Sheerin DATE April 22, 1981
SCALE

EACH ANALYSIS

EACH 6 (cont.)

Q_{P_2} = 8721 cfs stage = 15.4 ft.

The low point on Union Ave. would be overtopped by about 2.4 feet. A residential area on the north overbank and industrial complex on the south overbank would be subject to as much as 7 feet of backwater flooding. Excessive property damage and loss of life are possible.

In summary, about 25 residential homes and 3 industrial buildings would be subject to at least 7 feet of backwater flooding, resulting in the possible loss of many lives.

Accordingly, the dam at Framingham Reservoir No. 2 is classified as High Hazard.

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JOB FRAMINGHAM RES. NO. 2
SHEET NO. 18 OF 21
CALCULATED BY G. SHARRY DATE 9 APR 81
CHECKED BY H. Shevitz DATE April 27, 1981
SCALE

REACH ANALYSIS

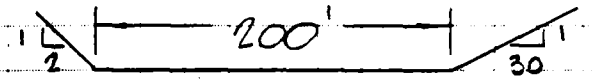
REACH 6 (cont.)

STAGE ABOVE CHANNEL INV (FT)	Q CULVERT (CFS)	Q WEIR (CFS)	Q TOTAL (CFS)
4	900		900
8	2565		2565
12	3825		3825
14	5535	299	5834
15	5895	1691	7586
16	6300	4661	10961
16.3	6400	5915	12315
16.5	6525	6852	13377

See rating curve, SH 21/21.

$Q_{P_1} = 9392$ cfs

stage = 15.6 ft.



TYP. X-SECTION - BACKWATER STORAGE
LOOKING UPSTREAM

$V_1 = \frac{\text{area (length)}}{43560} = \frac{7014(800)}{43560} = 128.8$ ac-ft $< \frac{1786}{2}$ ∴ OK

$Q_{P_2\,(TRIAL)} = Q_{P_1}\left(1 - \frac{V_1}{5}\right) = 9392\left(1 - \frac{128.8}{1786}\right) = 8715$ cfs

stage = 15.4 ft. $V_2 = \frac{6875(800)}{43560} = 126.3$ ac-ft

$V_{AVG} = 127.6$

$Q_{P_2} = Q_{P_1}\left(1 - \frac{V_{AVG}}{5}\right) = 9392\left(1 - \frac{127.6}{1786}\right) = \underline{8721}$ cfs

stage = 15.4 ft.

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JOB FRAMINGHAM RES. NO. 2
SHEET NO. 17 OF 21
CALCULATED BY G. STARRY DATE 8 APR 81
CHECKED BY H. Shevitz DATE April 27, 1981
SCALE

BREACH ANALYSIS

REACH 5 (cont.)

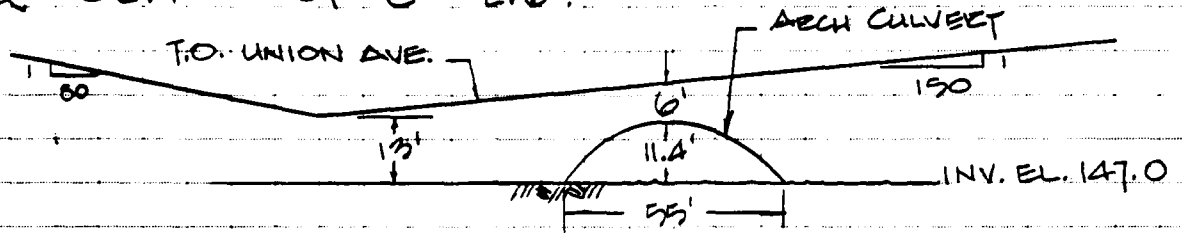
Q_{P_2} = 9392 cfs stage = 15.0 ft.

The low point on Franklin St. would be overtopped by about 3 feet. Also, the residential area just south of Maple St. (north overbank) would be inundated by backwater with depths of as much as 7 feet. Industrial buildings on the south overbank would also be subject to extensive backwater flooding. Reach 5 would be subject to a water surface increase of about 4.4 feet due to breach. Excessive property damage and loss of life are possible.

REACH 6

Downstream limit is Union Avenue. Length = 800 ft.

Union Ave. bridge will act as a dam. Develop rating curve at bridge. Arch culvert* has circle segment area = 432 ft². Use approximately equivalent rectangular area of 45' W x 9.5' H. Use FHA HEC-5 charts to rate flow through culvert assuming inlet control. For flow over road, use weir equation, $Q = CLH^{3/2}$ w/ C = 2.6.



ELEVATION LOOKING DOWNSTREAM

* Culvert dimensions from COE Dam Report No. MA00337.

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JOB FRAMINGHAM RES. NO. 2
SHEET NO. 16 OF 21
CALCULATED BY G. SHARZY DATE 8 APR 81
CHECKED BY H. Shoenfeld DATE April 27, 1981
SCALE

BREACH ANALYSIS

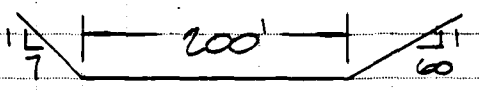
REACH 5 (cont.)

STAGE ABOVE CHANNEL INV (FT)	Q CULVERT (CFS)	Q WEIR (CFS)	Q TOTAL (CFS)
4	1000		1000
8	2900		2900
12	5200		5200
14	6150	956	7106
16	7000	5408	12408
16.5	7100	7260	14360
17	7400	9447	16847

See rating curve, SH 21/21.

$Q_{P_1} = 12189$ cfs

stage = 16.0 ft.



THR. X-SECTION - BACKWATER STORAGE
LOOKING UPSTREAM

$V_1 = \frac{\text{area (length)}}{435600} = \frac{11776\,(1600)}{435600} = 432.5$ ac-ft $< \frac{17806}{2}$ ∴ OK

$Q_{P_2(TRIAL)} = Q_{P_1}\left(1 - \frac{V_1}{5}\right) = 12189\left(1 - \frac{432.5}{17806}\right) = 9237$ cfs

stage = 15.0 ft $\quad V_2 = \frac{10538\,(1600)}{435600} = 387.1$ ac-ft.

$V_{AVG} = 409.8$ ac-ft

$Q_{P_2} = Q_{P_1}\left(1 - \frac{V_{AVG}}{5}\right) = 12189\left(1 - \frac{409.8}{17806}\right) = \underline{\underline{9392}}$ cfs

stage = 15.0 ft.

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JOB FRAMINGHAM DEES. NO. 2
SHEET NO. 15 OF 21
CALCULATED BY G. SHAIREY DATE 8 APR 81
CHECKED BY D. SHAVELTE DATE April 21, 1981
SCALE

BREACH ANALYSIS.

REACH 4 (cont.)

$Q_{P_2(TRIAL)} = 12142$ cfs stage = 18.8 ft.

$V_2 = \frac{14420(800)}{455100} = 2604.8$ ac-ft $V_{AVG} = 270.8$ ac ft

$Q_{P_2} = Q_{P_1}\left(1 - \frac{V_{AVG}}{S}\right) = 14368\left(1 - \frac{270.8}{1786}\right) = 12189$ cfs

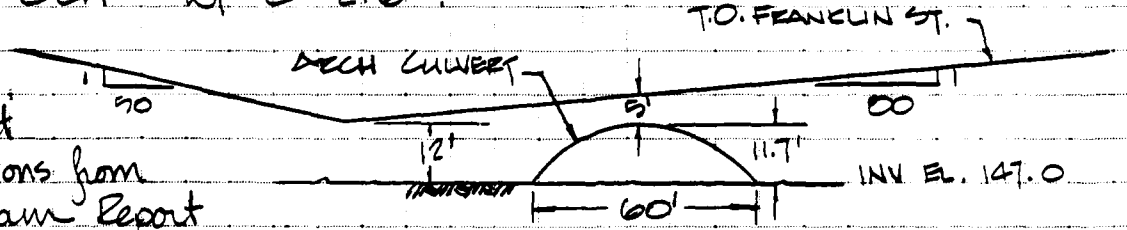
stage = 18.8 ft.

The railroad would be overtopped by about 0.8 feet. Minor damage could result.

REACH 5

Downstream limit is Franklin St. Length = 1600 ft.

Franklin St. bridge will act as a dam. Develop rating curve at bridge. Arch culvert* has circle segment area ≐ 482 ft². Use approximately equivalent rectangular area of 50' W x 9.5' H. Use FHA HEC-5 charts to rate flow through culvert assuming inlet control. For flow over road, use weir equation, $Q = CLH^{3/2}$ w/ C = 2.6.



ELEVATION LOOKING DOWNSTREAM

* Culvert dimensions from COE Dam Report No. MA00337.

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JOB FRAMINGHAM RES. NO. 2
SHEET NO. 14 OF 21
CALCULATED BY G. SHARRY DATE 8 APR 81
CHECKED BY D. Shoenfeld DATE April 27, 1981
SCALE

BREACH ANALYSIS

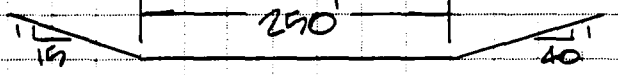
REACH 4 (cont.)

STAGE ABOVE CHANNEL INV (FT)	Q CULVERT (CFS)	Q WEIR (CFS)	Q TOTAL (CFS)
4	715		715
8	2103		2103
12	6090		6090
15	7830		7830
18	9400		9400
19	9740	3120	12860
19.5	9860	6209	16069
20	10150	10295	20445

See rating curve, SH 21/21.

$Q_{P_1} = 14368$ cfs

stage = 19.3 ft.



TYP. X-SECTION - BACKWATER STORAGE*
LOOKING UPSTREAM

$$V_1 = \frac{\text{area (length)}}{43560} = \frac{150(800)}{43560} = 2.76 \text{ ac ft} < \frac{17860}{2} \therefore \text{OK}$$

$$Q_{P_2\,(TRIAL)} = Q_{P_1}\left(1 - \frac{V_1}{S}\right) = 14368\left(1 - \frac{2.76}{17860}\right) = 12142 \text{ cfs}$$

* Ignore backwater at Eames Brook located southwest of railroad. A land bridge crosses the brook about 600 feet upstream of the confluence with the Sudbury River. Assume backflow would be held up long enough to allow breach flow to pass under and over railroad at culvert.

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JOB FRAMINGHAM RES. NO. 2
SHEET NO. 13 OF 21
CALCULATED BY G. SHARZU DATE 7APR81
CHECKED BY H. Shaevitz DATE April 27, 1981
SCALE

BREACH ANALYSIS

REACH 3 (cont.)

$$Q_{p_2} = Q_{p_1}\left(1 - \frac{V_{AVG}}{5}\right) = 16282\left(1 - \frac{2100}{1780}\right) = 14360 \text{ cfs}$$

stage = 11.7 ft.

No damage or loss of life would be expected along Reach 3.

REACH 4

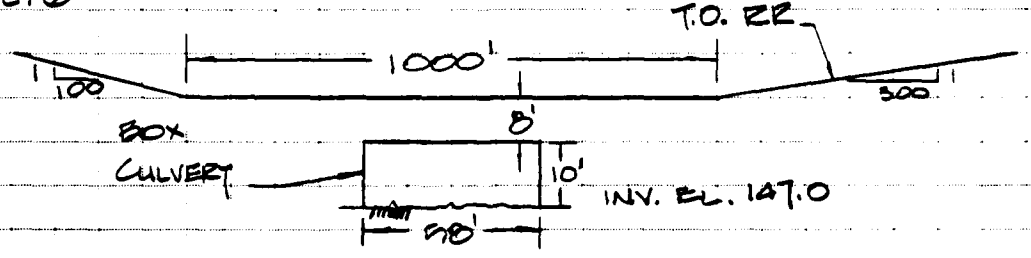
Downstream limit is New York, New Haven & Hartford Railroad bridge. Reach length = 800 ft.

RR bridge acts as a dam during high flood flows. Develop rating curve at bridge using FHA HEC-5 charts for culvert flow* at stages above 10 feet. For wide culvert, use Manning equation for low flows...

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \qquad n = 0.025$$

$S = 0.0005$

For flow over RR bridge, use weir equation, $Q = CLH^{3/2}$, w/ $C = 2.6$



ELEVATION LOOKING DOWNSTREAM ▽

* Inlet control
▽ Culvert dimensions from COE Dam Report No. MA00337, Page 11.

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JOB FRAMINGHAM RES. NO. 2
SHEET NO. 12 OF 21
CALCULATED BY G. SHARRY DATE 7 APR 81
CHECKED BY H. Shoenfeld DATE April 27, 1981
SCALE

BREACH ANALYSIS

REACH 3 (cont.)

Use Manning equation to develop rating curve for reach:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

Composite "n" = 0.06 $S = 0.0005$

STAGE ABOVE CHANNEL INV. (FT)	AREA (FT^2)	WETTED PERIMETER (FT)	Q (CFS)
3	810	390	732
6	2340	630	3117
9	4590	870	7719
12	7560	1110	15074
13	8710	1190	18221
14	9940	1270	21744

See rating curve, SH 21/21.

$Q_{P_1} = 16282$ cfs stage = 12.4 ft.

$V_1 = \frac{\text{area}(\text{length})}{43560} = \frac{8010(1200)}{43560} = 220.7$ ac-ft $< \frac{1786}{2}$ ∴ OK

$Q_{P_2(TRIAL)} = Q_{P_1}(1 - \frac{V_1}{S}) = 16282(1 - \frac{220.7}{1786}) = 14270$ cfs

stage = 11.7 ft. $V_2 = \frac{7231(1200)}{43560} = 199.2$ ac-ft

$V_{AVG} = 210.0$ ac-ft

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210 South Street
BOSTON, MASSACHUSETTS 02111
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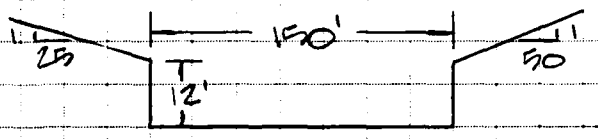
JOB FRAMINGHAM DES. NO. 2
SHEET NO. 11 OF 21
CALCULATED BY G. SHARRY DATE 7 APR 81
CHECKED BY H. SHOENFELD DATE April 27, 1981
SCALE

BREACH ANALYSIS

REACH 2 (cont.)

Length = 120 ft.



TYP. X-SECTION - BACKWATER STORAGE
LOOKING UPSTREAM

$Q_{p_1} = 16392$ cfs

Water surface elev. = 171.5, see rating curve, SH 20/21.

$V_1 = \frac{\text{area (length)}}{43560} = \frac{4359\,(120)}{43560} = 12.0$ ac-ft $< \frac{17.86}{2}$ ∴ OK

$Q_{P_{2(TRIAL)}} = Q_{p_1}\left(1 - \frac{V_1}{3}\right) = 16392\left(1 - \frac{12.0}{17.86}\right) = 16282$ cfs

water surface el. = 171.5 $V_2 = \frac{4359\,(120)}{43560} = 12.0$ ac-ft

$V_{AVG} = 12.0$ ac-ft

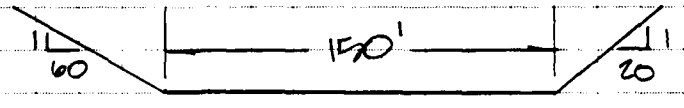
$Q_{p_2} = Q_{p_1}\left(1 - \frac{V_{AVG}}{3}\right) = 16392\left(1 - \frac{12.0}{17.86}\right) = \underline{16282}$ cfs

Water surface el. = 171.5 NGVD

Winter St. would be overtopped by about 3.5 feet, an increase of about 8.5 feet over antecedent conditions. Appreciable damage and loss of life are possible.

REACH 3

Length = 1200 ft



TYPICAL X-SECTION
LOOKING DOWNSTREAM

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JOB FRAMINGHAM CES. NO. 2
SHEET NO. 10 OF 21
CALCULATED BY G. SHAREY DATE 7 APR 81
CHECKED BY H. Shevitz DATE April 27, 1981
SCALE

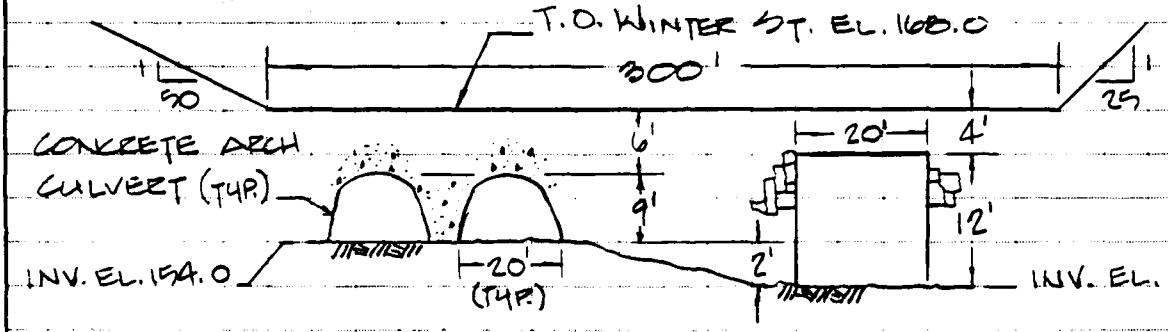
BREACH ANALYSIS

REACH 2

Downstream limit is Winter St.

Winter St. bridge will act as a dam. Develop flow rating at Winter St. bridge. For culvert flow, use FHA HEC-5 charts assuming inlet control. At low stages, use Manning equation, $Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$

with $n = 0.018$, $S = 0.0001$... For flow over road, use weir equation, $Q = CLH^{3/2}$ with $C = 2.6$.



ELEVATION LOOKING DOWNSTREAM

W.S. ELEV. ABOVE NGVD (FT)	Q ARCH CULV's (CFS)	Q BOX CULV. (CFS)	Q WEIR (CFS)	Q TOTAL (CFS)
156	93	214		307
160	331	597		928
162	2500	820		3320
164	3250	2100		5350
166	3800	2600		6400
168	4400	3160		7560
170	5000	3600	2758	11358
171	5160	3760	5573	14493
172	5400	4000	9360	18760

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JOB FRAMINGHAM RES. NO. 2
SHEET NO 9 OF 21
CALCULATED BY G. SHARRY DATE 27 MAY 81
CHECKED BY [illegible] DATE [illegible]
SCALE

BREACH ANALYSIS

BREACH 1 (cont.)

$V_1 = 1/3\,\pi(11.3)(2033^2 + 2274^2 + 2033(2274)) = 3783$ ac-ft

3783 ac ft $\gg$ 1786/2 ; also, a stage of 132.7 would cause a backwater on Res. No. 2 upstream.

Try $V_1 = (1)S_{initial} = 1786$ ac-ft ; this storage volume would result in a stage of 176.7 on Res. No. 1.

$S_{(available\ at\ No.2)} = S_{179.5} - S_{176.7} = 2800 - 2000 = 500$ ac-ft
ac-ft < 1786 ∴ not possible.

Try $V_1 = 3/4\,S_{initial} = 1340$ ac-ft stage = 175.4
$S_{available} = 1130$ ac-ft < 1340 ac-ft ∴ NG.

Try $V_1 = 0.7\,S_{initial} = 1250$ ac-ft stage = 175.1
$S_{available} = 1213$ ac-ft < 1250 ac-ft ∴ NG.

Try $V_1 = 2/3\,S_{initial} = 1191$ ac-ft stage = 175.0
$S_{available} = 1240$ ac-ft > 1191 ac-ft ∴ OK

Use $V_1 =$ [illegible] ac-ft stage = 175.0

$Q_{out} = Q_{P.M.F.}\left(1 - \frac{V_1}{1786}\right) = 49205\left(1 - \frac{1191}{1786}\right) = 16392$ cfs

Check: [illegible] curve, p. D-4, COE Dam Report No. 1A, [illegible]; stage @ Q = 16392 = 175.0.
175.0 = stage which is due to V_1, ∴ OK.

This breach would be overtopped by [illegible] feet of water at No. 2. Associated storage [illegible] ac-ft.

NOT AVAILABLE AT THIS TIME

END

FILMED

7-85

DTIC